This is similar to the classification in Table 6-5 of FEMA-273, except that $\mu_\Delta = 5$ is used as the threshold for high ductility, rather than $\mu_\Delta = 4$. The less conservative value of five is considered more appropriate for damage evaluation, as opposed to retrofit design. The value of five also correlates best with the data for the shear strength recommendations of Section 5.3.6.b.

5.3.5 Moment Strength

The moment strength of a reinforced concrete component under flexure and possible axial loads is calculated according to conventional procedures, as defined in ACI-318, Section 10.2 (ACI, 1995), except that expected material strengths are used as discussed in Section 5.3.2 of this document. The moment strength accounts for all reinforcement that contributes to flexural strength. For example, the moment strength for a wall pier (component type RC1 or RC2) includes all well-anchored vertical bars at the section of interest, not just those in the wall boundaries. The axial load present on the wall component is taken into account in the calculation of moment strength.

For wall components that experience significant earthquake axial loads, such as the piers in a coupled wall system, the moment strength in each direction must consider the axial load combination corresponding to moments in that direction.

For sections with an overall reinforcement ratio, ρ_g, less than 0.008 x (60ksi/f_y) the expected cracking moment strength, M_{cr}, may exceed the expected moment strength M_e. In such a case, both M_e and M_{cr} are considered in determining the governing mechanism and behavior mode.

a. Uncertainties or discrepancies in strength

Typically, there should be little uncertainty in the calculation of moment strength for a reinforced concrete component if reinforcement sizes, layout, and the steel and concrete material strengths have been established. The possible range of axial load on the component must also be considered.

b. Effective Flange Width

When wall sections have flanges or returns, the moment strength includes the effective width of flanges that contribute to flexural strength. C-shaped, I-shaped, L-shaped, T-shaped, and box-shaped wall sections fall in this category. The effective flange width is a function of the moment-to-shear ratio (*M*/*V*) for the wall component. Moment strength is relatively insensitive to the assumed flange width in compression, but can be quite sensitive to the assumed flange width in tension. Underestimating the effective flange width could lead to a conclusion that a wall is flexure-critical when in reality it is shear-critical. Typically, as displacement (or ductility level) increases, more of the vertical reinforcement in the flange is mobilized to resist flexure, and the effective flange width increases.

For isolated (cantilever) walls, effective flange width can be related to wall height, h_w, as described and illustrated in Section 5.22 of Paulay and Priestley (1992). For wider applicability to different loading patterns, the moment-to-shear ratio (M/V) can be used in place of the wall height.

FEMA 273 and ATC-40 prescribe an effective flange width of one-quarter of the wall height on each side of the wall web, with engineering judgment to be exercised if significant reinforcement is located outside this width. The 1997 UBC prescribes an effective flange width on each side of the wall web of 0.15 times the wall height. The proposed *NEHRP Provisions for New Buildings* (BSSC, 1997) prescribe a maximum effective width on each side of the wall web of 0.15 times the wall height for compression flanges and 0.30 times the wall height for tension flanges.

A more specific estimate of effective flange width is supported by research (Paulay and Preistley, 1992; Wallace and Thomsen, 1995) and is recommended in this document as defined below:

The effective flange width in compression, on each side of the wall web, may be taken as 0.15 times the moment-to-shear ratio (*M*/*V*). The effective flange width in tension, on each side of the wall web, may be taken as 0.5 to 1.0 times the moment-to-shear ratio (*M*/*V*). The effective width of the flange does not exceed the actual width of the flange, and the assumed flange widths of adjacent parallel walls do not overlap.

The foundation structure should be checked to ensure that the uplift forces in tension flanges can be developed.

c. Contribution of Frame and Slab Coupling to Wall Capacity

Beams and slabs that frame into a wall may contribute to the lateral capacity of the structural system. This was demonstrated in the testing of a full-scale seven-story wall structure in Japan (Wight, 1985). Beams transverse to the wall and in-line with the wall helped resist the lateral displacement of the wall, resulting in a total strength significantly greater than that of the wall alone.

5.3.6 Shear Strength

a. Shear Demand and Capacity

Consistent with the requirement in Section 2.4 to identify the mechanism of inelastic lateral response for the structure, shear demand is based on the expected strength developed at the locations of nonlinear action (e.g., plastic hinge zones). This is also addressed in Section 6.4.1.1 of FEMA 273.

For behavior modes with intermediate ductility capacity such as flexure/diagonal tension, flexure/diagonal compression, and flexure/sliding shear, the shear demand is based on the expected moment strength developed in the plastic hinge regions. The shear demand so derived can be magnified because of inelastic dynamic effects which change the pattern of inertial force in the building from the inverted triangular distribution typically assumed in analysis and design.

For the example of a cantilever wall with a plastic hinge at the base, the shear demand will equal the expected moment strength at the base divided by 2/3 the wall height for an inverted triangular distribution of lateral forces. However, if inelastic dynamic effects cause the pattern of lateral forces to approach a uniform distribution, then the shear demand will increase to a value equal to the expected moment strength at the base (which will still be developed) divided by 1/2 the wall height.

Inelastic dynamic effects have been studied by researchers, and a shear magnification factor, ω_v, taken as a function of the number of stories, is recommended by Paulay and Priestley (1992). The dynamic amplification of shear demand can be considered by use of such a factor or by considering different vertical distributions of lateral forces in the nonlinear static analysis.

Traditional design equations for shear strength tend to reflect the lower bound of test results, but the overall correlation of the equations with the data is not good. While some wall specimens show strength values close to the prediction of design equations, others show strength values five times higher than the predicted values (Cardenas, 1973).

b. Diagonal Tension

FEMA 273 specifies that the shear strength of reinforced concrete walls be calculated according to Section 21.6 of ACI 318-95. The applicable ACI equations are:

$V_n = A_{cv}\,(2\sqrt{f'_{ce}} + \rho_n f_{ye})$ for walls with a ratio of h_w / l_w greater than 2.0, and

$V_n = A_{cv}\,(3\sqrt{f'_{ce}} + \rho_n f_{ye})$ for walls with a ratio of h_w / l_w less than 1.5

FEMA 273 allows the use of these equations for walls with reinforcement ratios, ρ_n, as low as 0.0015 — below the 1995 ACI-specified minimum of 0.0025. For walls with reinforcement ratios below 0.0015, FEMA 273 specifies that the strength calculated at $\rho_n = 0.0015$ can still be used.

ATC-40 modifies the provisions of FEMA 273 and ACI 318-95 for wall shear strength. The principal modifications are that V_n need not be taken lower than $4\sqrt{f'_{ce}}\,A_{cv}$, and that $2\sqrt{f'_{ce}}$ is assumed for the concrete contribution to shear strength, regardless of the ratio of h_w / l_w. Reinforcement ratios less than 0.0025 are also addressed differently in the ATC-40 document, but in typical cases of light reinforcement, the $4\sqrt{f'_{ce}}\,A_{cv}$ lower limit governs the calculations.

The FEMA 273 and ATC-40 wall shear strength recommendations are design equations that do not explicitly consider:

- The effect of axial load on shear strength
- The distinction between shear strength at plastic-hinge zones versus that away from plastic-hinge zones
- The potential degradation of shear strength at plastic hinge zones

Equations for wall shear strength given in Paulay and Priestley (1992) recognize a significant increase in shear strength due to axial load level. The equations also recommend a much lower shear strength at plastic-hinge zones, accounting for potential degradation, than away from plastic-hinge zones.

If warranted by the specific conditions under evaluation, an approach similar to that used by Priestley et al. (1996) and Kowalsky et al. (1997) for columns can be used. The following shear strength equation:

$$V_n = V_c + V_s + V_p \tag{5-1}$$

expresses the shear strength as the sum of three components: the contributions of the concrete, steel, and axial load. Each of these components is defined as follows:

$$V_c = \alpha\beta k_{rc}\sqrt{f'_{ce}}\; b_w(0.8l_w) \tag{5-2}$$

where k_{rc} is a function of ductility, as shown below:

$k_{rc} = 3.5$ for low ductility ($\mu_\Delta \leq 2$) and away from plastic hinge regions.

$k_{rc} = 0.6$ for high ductility ($\mu_\Delta \geq 5$)

For values of ductility between the above limits, k_{rc} is calculated by linear interpolation.

The coefficient α accounts for wall aspect ratio, as considered in the ACI-318 equations:

$$\alpha = 3 - M/(0.8l_wV) \tag{5-3}$$

$$1.0 \leq \alpha \leq 1.5$$

The coefficient β accounts for longitudinal reinforcement ratio, as recognized by ASCE/ACI Task Committee 426 (1973):

$$\beta = 0.5 + 20\rho_g \tag{5-4}$$

$$\beta \leq 1.0$$

where ρ_g is the ratio of total longitudinal reinforcement over gross cross-sectional area for the wall component.

$$V_s = \rho_n f_{ye}\, b_w h_d \tag{5-5}$$

where h_d equals the height over which horizontal reinforcement contributes to shear strength, taken as $(l_w - c)\cot\theta$, where θ equals the angle, from the vertical, of the critical inclined shear crack. θ is taken as 35 degrees unless limited to larger angles by the potential corner-to-corner crack. Thus h_d does not exceed the clear height of a wall pier.

$$V_p = ((l_w - c)N_u)/(2M/V) \tag{5-6}$$

M/V is taken as the larger of the values at the top and bottom of the wall pier. Thus $2M/V$ should not be less than the clear height of the wall pier.

These shear strength equations might also apply to coupling beams, for which l_w is the overall depth (measured vertically) of the coupling beam, and h_d is the horizontal length over which vertical stirrups contribute to shear strength.

c. Diagonal Compression (Web Crushing)

Walls and wall piers that have sufficient horizontal reinforcement to prevent a shear failure in diagonal tension may still suffer a shear failure associated with diagonal compression or web crushing. Web crushing behavior becomes more likely at higher levels of lateral deformation, and for walls with higher axial loads, N_u.

The web-crushing shear strength of a wall can be estimated according to the following equation (Oesterle et al., 1983):

$$V_{wc} = \frac{1.8f'_{ce}}{1+\left(600-2000\dfrac{N_u}{A_g f'_{ce}}\right)\delta}\, b_w(0.8l_w) \tag{5-7}$$

where δ is the story drift ratio to which the wall component is subjected. The above equation applies to a typical range of axial loads for walls: $0 < N_u / A_g\, f'_{ce} < 0.09$. For walls with higher axial loads, V_{wc} is held constant at the value calculated for $N_u / A_g\, f'_{ce} = 0.09$. Thus, V_{wc} does not exceed:

$$V_{wc} = \frac{1.8f'_{ce}}{1+420\delta}\, b_w(0.8l_w) \tag{5-8}$$

The above expressions give a lower bound to the test data. Multiplying V_{wc} by 1.5 would give a reasonable upper bound to the web-crushing shear strength.

An alternative expression for the web-crushing shear strength is given in Section 5.44 of Paulay and Priestley (1992). This expression is based on displacement ductility rather than story drift and does not consider the effect of axial load.

The above procedures apply to the flexure/web-crushing behavior mode, and they indicate a degradation of web-crushing strength with increasing drift or ductility. Tests (Barda et al., 1976) have also shown preemptive web-crushing behavior; that is, web crushing that occurs at small displacement levels, before the wall has attained its flexural strength. The test results show that walls may suffer preemptive web crushing when shear stress levels exceed $12\sqrt{f'_{ce}}$ to $15\sqrt{f'_{ce}}$.

d. Sliding Shear

Sliding shear strength is assessed at construction joints and plastic hinge zones using the shear friction provisions of Section 11.7.4 of ACI 318-95. All reinforcement that crosses the potential sliding plane and is located within the wall section that resists shear is assumed to contribute to the sliding-shear strength.

Isolated Walls and Wall Piers. For isolated walls and wall piers, the potential sliding plane is a horizontal plane. Vertical reinforcement that crosses this plane and contributes to flexural strength also contributes to sliding-shear strength.

Shear transfer occurs primarily in the web of a wall section rather than in wall flanges. All vertical bars located in the web of the wall section, or within a distance b_w from the web, are considered effective as shear-friction reinforcement. For wall sections that have typical columns as boundary elements, the vertical bars in the wall web plus those in the boundary elements can be used for shear friction. For wall sections that have wide flanges as boundary elements, the vertical bars placed in the flanges, at a distance of more than b_w from the web, are not considered effective for shear friction (Paulay and Priestley, 1992).

It may be argued that only the reinforcement on the tension side of the neutral axis should be effective in contributing to shear friction strength, but such a recommendation has not been well established or tested.

Sliding-shear strength is investigated at construction joints and at plastic-hinge zones. The quality of the construction joint should be considered in establishing the appropriate coefficient of friction, μ, as specified in ACI 318. At plastic-hinge regions, increasing cyclic deformations cause horizontal flexural cracks at the potential sliding plane to open more widely, which results in a degradation of sliding-shear strength. In such a case, the effective coefficient of friction, μ, can be considered to be reduced.

A more detailed assessment of the sliding-shear strength of squat walls can be carried out according to the recommendations in Section 5.7 of Paulay and Priestley (1992).

Coupling Beams. If diagonal tension failures are prevented by sufficient stirrup reinforcement, and if diagonal bars are not used, sliding shear is likely to occur in short coupling beams at moderate-to-high ductilities. According to Paulay and Priestley (1992), there is a danger of sliding shear occurring in coupling beams whenever V_u exceeds $1.2\ (l_n / h)\sqrt{f'_{ce}}\, b_w\, d$, (assuming diagonal bars are not present and stirrups prevent a diagonal tension failure). The provisions of the 1997 UBC require diagonal bars in coupling beams when V_u exceeds $4\sqrt{f'_{ce}}\, b_w d$ and l_n/d is less than four.

For this document, in the absence of more detailed analyses, the sliding-shear strength of coupling beams may be assumed to be equal to $1.2\ (l_n / h)\sqrt{f'_{ce}}\, b_w$ d at high ductility levels and may be assumed equal to $3(l_n/h)\sqrt{f'_{ce}}\, b_w d$ at moderate ductility levels. Alternatively, a shear-friction approach could be considered for coupling beams.

5.3.7 Wall Boundary Confinement

For walls responding in flexure, boundary-tie reinforcement is usually needed in the plastic hinge regions to allow high ductility values to be achieved. Table 6-18 of FEMA 273 and Table 9-10 of ATC-40 reference the boundary confinement requirements of ACI 318-95, and both FEMA 273 and ATC-40 reference the 1994 Uniform Building Code (ICBO, 1994) and Wallace (1994, 1995). These references give substantially different recommendations for boundary tie requirements.

Paulay and Priestley (1992) and the New Zealand concrete code (SANZ, 1995) present more widely applicable recommendations for wall boundary ties. An adaptation of these recommendations is given below.

For walls to achieve *high ductility capacities*, boundary ties must meet the following criteria:

a. Walls with $c \le 0.15 l_w$ and $\rho_l \le 400 / f_{ye}$:

   Boundary ties are not required.

b. Walls with $c \le 0.15 l_w$ and $\rho_l > 400 / f_{ye}$:

   Boundary ties are necessary, as specified below, to prevent buckling of longitudinal bars:

   - Boundary ties extend over a length of the wall section at the compression boundary greater than or equal to c', taken as the larger of $c - 0.1 l_w$ or $0.5c$, where c is the distance from the compression face to the neutral axis.
   - Boundary ties extend over a height of the wall at the plastic hinge region greater than or equal to $2l_p$.
   - Ties are spaced at no more than $6d_b$, where d_b is the diameter of the longitudinal bar being tied.
   - Each longitudinal bar is restrained against bar buckling by either a crosstie or a 90-degree bend of a hoop with d_{bt} greater than or equal to $0.25d_b$; or is restrained by a hoop leg parallel to the wall surface which spans not more than 14 in. between 90-degree bends of the hoop, with d_{bt} greater than or equal to $0.4d_b$. (d_{bt} is the diameter of the crosstie or hoop.)

c. Walls with $c > 0.15 l_w$:

   Boundary ties are necessary to prevent buckling of longitudinal bars and to confine the concrete to achieve higher compressive strains. In addition to meeting the requirements of item (b) above, ties are provided so that:

$$A_{sh} \ge 0.2 s h_c \left(\frac{f'_{ce}}{f_{yhe}}\right)\left(\frac{A_g}{A_{ch}}\right)\left(\frac{c}{l_w} - 0.10\right) \qquad (5\text{-}9)$$

The term ρ_l is the local reinforcement ratio for flexural reinforcement, as defined below:

$$\rho_l = A_s / b s_l$$

where A_s is the area of vertical wall reinforcement in a layer spaced at s_l along the length of the wall, and where b is the width of the wall at the compression boundary.

Walls that do not meet the criteria for high ductility capacities, but which have some boundary ties in the plastic hinge region, spaced at no more that 10 d_b, and that have dimensions $c \le 0.20 l_w$, can be assumed to achieve *moderate ductility capacities* $(2 \le \mu_\Delta \le 5)$.

5.3.8 Lap Splice Strength

As specified in Section 6.4.5 of FEMA 273 and Section 9.5.4.5 of ATC-40, the strength of existing lap splices may be estimated according to the ratio of lap-length provided to the tension development length required by ACI 318-95.

Thus, the strength of lap splices can be taken as:

$$f_s = (l_b / l_d) f_{ye} \qquad (5\text{-}10)$$

where:

	f_s	=	stress capacity of the lap splice
	l_b	=	provided lap-splice length
	l_d	=	tension development length for straight bars, taken according to ACI 318, Chapter 12

Note that the tension development length, l_d, is used in the above equations without the 1.3 splice factor of ACI-318, because the specified lap-splice lengths prescribed for new design are conservative (ATC 1996).

For splices in plastic-hinge regions, the evaluation should consider that lap-splice slip may still be possible even if splice lengths are adequate according to the above criteria.

A method of assessing lap-splice strength and the ductility capacity of flexural plastic hinges that contain lap splices is given in Sections 5.5.4, 7.4.5, and 7.4.6 of Priestley et al. (1996). The method allows the calculation of strength based on a fundamental consideration of the mechanics of lap-splice slip.

When the lap-splice strength is less than that required to yield the reinforcement, the full moment strength of the section will not develop. Even when lap splices have sufficient capacity to yield the reinforcement, they may still slip when moderate ductility levels are reached. As developed for columns, the Priestley et al. (1996) method indicates that all lap splices may become prone to slipping when the concrete compressive strain reaches 0.002. The method gives an estimate of the degradation of lap splice strength with increasing ductility, which results in a loss of moment capacity down to a residual value based on axial force alone.

5.3.9 Wall Buckling

Thin wall sections responding in flexure may be prone to out-of-plane buckling, typically at higher ductility levels. The 1997 UBC prescribes a minimum wall thickness of 1/16 the clear story height for walls that require boundary confinement. Out-of-plane buckling is possible in plastic-hinge regions of walls even if they do not require confinement. Paulay and Priestley (1992, 1993) address the wall buckling phenomenon in detail, and the New Zealand concrete code (SANZ, 1995) provides design recommendations for minimum wall thickness based on the research.

Flanged or barbell-shaped wall sections are typically not vulnerable to buckling, unless the flange is unusually narrow, having a width, b, less than that specified below.

Based on the research, the following simplified criteria are recommended: Walls with width, b, equal to or greater than $l_u/16$ can be assumed to achieve high ductility capacity without buckling. Walls with b equal to $l_u/24$ can be assumed to be vulnerable to buckling at moderate-to-high ductility levels.

The length, l_u, is taken as the smaller of:

- The clear story height between floors bracing the wall in the out-of-plane direction, *and*
- $2.5l_p$ for single-curtain walls and walls with ρ_l greater than $200/f_{ye}$, *or* $2.0l_p$ for two-curtain walls with ρ_l less than or equal to $200/f_{ye}$.

The term b is the width of the wall at the compression boundary. The term ρ_l is the local reinforcement ratio for flexural reinforcement, as defined in Section 5.3.7.

FEMA 273 and ATC-40 do not address overall wall buckling.

5.4 Symbols for Reinforced Concrete

Symbols that are used in this chapter are defined below. Further information on some of the variables used (particularly those noted "per ACI") may be found by looking up the symbol in Appendix D of ACI 318-95.

A_{ch} = Cross sectional area of confined core of wall boundary region, measured out-to-out of confining reinforcement and contained within a length c' from the end of the wall, Section 5.3.7

A_{cv} = Net area of concrete section bounded by web thickness and length of section in the direction of shear force considered, in^2 (per ACI)

A_g = Gross cross sectional area of wall boundary region, taken over a length c' from the end of the wall, Section 5.3.7

A_{sh} = Total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension h_c. (per ACI)

b = Width of compression face of member, in (per ACI)

b_w = Web width, in (per ACI)

c = Distance from extreme compressive fiber to neutral axis (per ACI)

c' = Length of wall section over which boundary ties are required, per Section 5.3.7

d_b = Bar diameter (per ACI)

d_{bt} = Bar diameter of tie or loop

f'_c = Specified compressive strength of concrete, psi (per ACI)

f'_{ce} = Expected compressive strength of concrete, psi

f_y = Specified yield strength of nonprestressed reinforcement, psi. (per ACI)

f_{ye} = Expected yield strength of nonprestressed reinforcement, psi.

f_{yh} = Specified yield strength of transverse reinforcement, psi (per ACI)

f_{yhe} = Expected yield strength of transverse reinforcement, psi

h_c = Cross sectional dimension of confined core of wall boundary region, measured out-to-out of confining reinforcement

h_d = Height over which horizontal reinforcement contributes to V_s per Section 5.3.6.b

h_w = Height of wall or segment of wall considered (per ACI)

k_{rc} = Coefficient accounting the effect of ductility demand on V_c per Section 5.3.6.b

l_p = Equivalent plastic hinge length, determined according to Section 5.3.3.

l_u = Unsupported length considered for wall buckling, determined according to 5.3.9

l_n = Beam clear span (per ACI)

l_w = Length of entire wall or segment of wall considered in direction of shear force (per ACI). (For isolated walls and wall piers equals horizontal length, for spandrels and coupling beams equals vertical dimension i.e., overall depth)

M_{cr} = Cracking moment (per ACI)

M_e = Expected moment strength at section, equal to nominal moment strength considering expected material strengths.

M_n = Nominal moment strength at section (per ACI)

M_u = Factored moment at section (per ACI)

M/V = Ratio of moment to shear at a section. When moment or shear results from gravity loads in addition to seismic forces, can be taken as M_u/V_u

N_u = Factored axial load normal to cross section occurring simultaneously with V_u; to be taken as positive for compression, negative for tension (per ACI)

s = Spacing of transverse reinforcement measured along the longitudinal axis of the structural member (per ACI)

s_l = spacing of vertical reinforcement in wall (per ACI)

V_c = Nominal shear strength provided by concrete (per ACI)

V_n = Nominal shear strength (per ACI)

V_p = Nominal shear strength related to axial load per Section 5.3.6

V_s = Nominal shear strength provided by shear reinforcement (per ACI)

V_u = Factored shear force at section (per ACI)

V_{wc} = Web crushing shear strength per Section 5.3.6.c

α = Coefficient accounting for wall aspect ratio effect on V_c per Section 5.3.6.b

β = Coefficient accounting for longitudinal reinforcement effect on V_c per Section 5.3.6.b

δ = Story drift ratio for a component, corresponding to the global target displacement, used in the computation of V_{wc}, Section 5.3.6.c

μ = Coefficient of friction (per ACI)

μ_Δ = Displacement ductility demand for a component, used in Section 5.3.4, as discussed in Section 6.4.2.4 of FEMA-273. Equal to the component deformation corresponding to the global target displacement, divided by the effective yield displacement of the component (which is defined in Section 6.4.1.2B of FEMA-273).

ρ_g = Ratio of total reinforcement area to cross-sectional area of wall.

ρ_l = Local reinforcement ratio in boundary region of wall according to Section 5.3.7

ρ_n = Ratio of distributed shear reinforcement on a plane perpendicular to plane of A_{cv} (per ACI). (For typical wall piers and isolated walls indicates amount of horizontal reinforcement.)

5.5 Reinforced Concrete Component Guides

The following Component Damage Classification Guides contain details of the behavior modes for reinforced concrete components. Included are the distinguishing characteristics of the specific behavior mode, the description of damage at various levels of severity, and performance restoration measures. Information may not be included in the Component Damage Classification Guides for certain damage severity levels; in these instances, for the behavior mode under consideration, it is not possible to make refined distinctions with regard to severity of damage. See also Section 3.5 for general discussion of the use of the Component Guides and Section 4.4.3 for information on the modeling and acceptability criteria for components.

RC1A	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Concrete**
		Component Type:	**Isolated Wall or Stronger Pier**
		Behavior Mode:	**Ductile Flexural**

How to distinguish behavior mode:

By observation:

Wide flexural cracking and spalling should be concentrated in the plastic hinge zone, although minor flexural cracking (width not exceeding 1/8 in.) may extend beyond the plastic hinge zone. Shear cracks may occur but widths should not exceed 1/8 in. If cracks exceed this width, see RC1B. Vertical cracks and spalling may occur at the extreme fibers of the plastic hinge region (toe region). If there is spalling or crushing of concrete within the web or center area of the section, see RC1C. If reinforcing bars in the toe region buckle, see RC1E.

Ductile flexural behavior typically occurs in well-designed walls that have sufficient horizontal reinforcement and do not have heavy vertical (flexural) reinforcement.

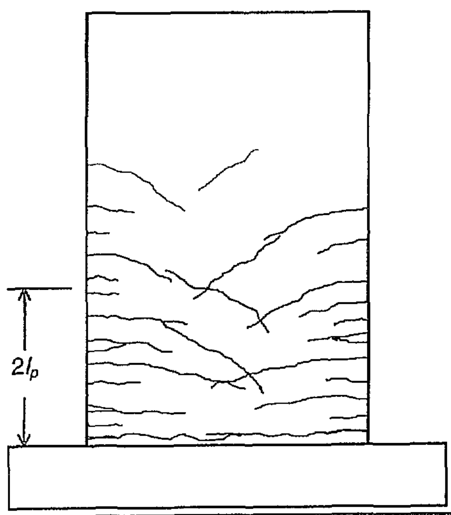
Note: At low damage levels, damage observations will be similar to those for other behavior modes.

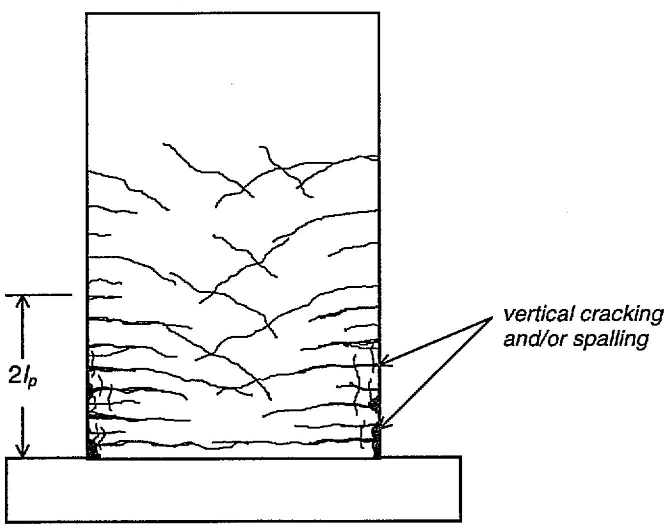
By analysis:

Strength in all other behavior modes, even after possible degradation, is sufficient to ensure that flexural behavior controls. Strength associated with shear, web crushing, sliding shear, and lap splices — taken for conditions of high ductility — exceeds moment strength. Foundation rocking strength exceeds moment strength. Boundary ties are sufficient to prevent bar buckling or loss of confinement, and wall thickness is sufficient to prevent overall buckling.

Refer to Evaluation Procedures for:

- Identifying plastic hinge locations and extent.
- Identifying flexural versus shear cracks.
- Calculation of moment, diagonal tension, web-crushing, sliding-shear, lap splice, and foundation rocking strength.
- Required boundary ties and wall thicknesses.

Severity	**Description of Damage**	**Performance Restoration Measures**
Insignificant $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No crack widths exceed 3/16 in., <u>and</u> • No shear cracks exceed 1/8 in., <u>and</u> • No significant spalling or vertical cracking *Typical Appearance:*  Note: *l_p is length of plastic hinge. See Section 5.3.3*	(Repairs may be necessary for restoration of nonstructural characteristics.)

COMPONENT DAMAGE CLASSIFICATION GUIDE **continued**		**RC1A**
Severity	**Description of Damage**	**Performance Restoration Measures**
Slight $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • Crack widths do not exceed 1/4 in., and • No shear cracks exceed 1/8 in., and • No significant spalling or vertical cracking, and • No buckled or fractured reinforcement, and • No significant residual displacement. *Typical Appearance:* Similar to insignificant damage, except wider flexural cracks and typically more extensive cracking.	• Inject cracks $\lambda_K{}^* = 0.9$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Moderate $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Criteria:* • Spalling or vertical cracking (or incipient spalling as identified by sounding) occurs at toe regions in plastic hinge zone, typically limited to the cover concrete, and • No buckled or fractured reinforcement, and • No significant residual displacement. *Typical Appearance:* Crack widths typically do not exceed 1/4 in.  Note: *l_p is length of plastic hinge. See Section 5.3.3*	• Remove and patch spalled and loose concrete. Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Heavy	Not Used	
Extreme	*Criteria:* • Reinforcement has fractured. *Typical Indications* • Wide flexural cracking typically concentrated in a single crack. • Large residual displacement.	• Replacement or enhancement required.

RC1B	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Concrete**
		Component Type:	**Isolated Wall or Stronger Pier**
		Behavior Mode:	**Flexure/Diagonal Tension**

How to distinguish behavior mode:

By observation:
For insignificant to moderate levels of damage, indications will be similar to those for RC1A, although shear cracking may begin at lower ductility levels. At higher levels of damage, one or more wide shear cracks begin to form.

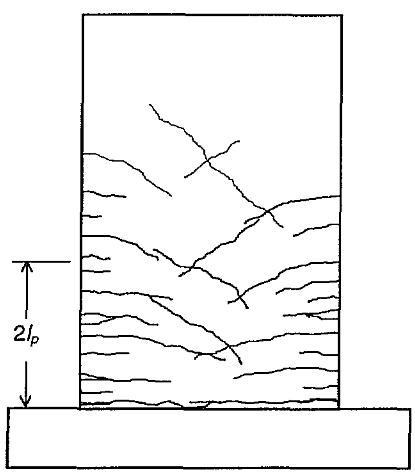
Typically occurs in walls that have a low-to-moderate amount of horizontal reinforcement, and which may have heavy vertical (flexural) reinforcement. May be most prevalent in walls with intermediate aspect ratios, $M/Vl_w \approx 2$, but depending on the reinforcement, can occur over a wide range of aspect ratios.

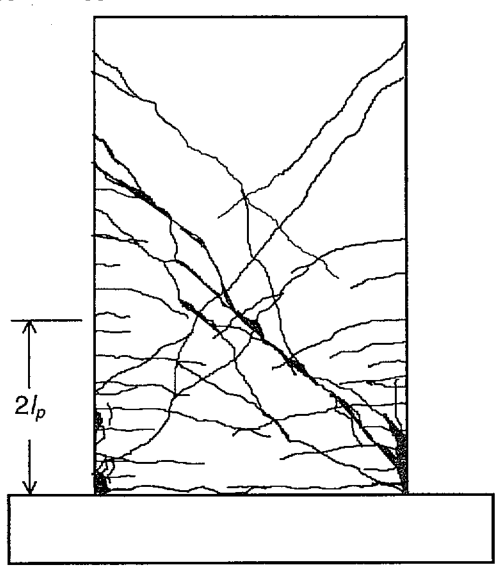
By analysis:
Shear strength calculated for conditions of low ductility exceeds flexural capacity, but shear strength calculated for conditions of high ductility is less than the flexural capacity.

Foundation rocking strength exceeds moment strength. Boundary ties are sufficient to prevent buckling of longitudinal bars and loss of confinement prior to shear failure. Wall thickness is sufficient to prevent overall buckling prior to shear failure. Sliding shear strength is not exceeded.

Refer to Evaluation Procedures for:

- Identifying plastic hinge locations and extent.
- Identifying flexural versus shear cracks.
- Calculation of moment, diagonal tension, web-crushing, sliding-shear, lap splice, and foundation rocking strength.
- Required boundary ties and wall thickness.

Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • Shear crack widths do not exceed 1/16 in., and • Flexural crack widths do not exceed 3/16 in., and • No significant spalling or vertical cracking. *Typical Appearance:*  Note: *l_p is length of plastic hinge. See Section 5.3.3*	(Repairs may be necessary for restoration of nonstructural characteristics.)

COMPONENT DAMAGE CLASSIFICATION GUIDE **continued**		**RC1B**
Severity	**Description of Damage**	**Performance Restoration Measures**
Slight	Not Used	
Moderate $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Criteria:* • Shear crack widths do not exceed 1/8 in., <u>and</u> • Flexural crack widths do not exceed 1/4 in., <u>and</u> • Shear cracks exceed 1/16 in., <u>or</u> limited spalling (or incipient spalling as identified by sounding) occurs at web or toe regions, <u>and</u> • No buckled or fractured reinforcement, <u>and</u> • No significant residual displacement. *Typical Appearance:* Similar to insignificant damage except wider cracks, possible spalling, and typically more extensive cracking.	• Remove and patch spalled and loose concrete. Inject cracks. $\lambda_K^* = 0.8$ $\lambda_Q^* = 1.0$ $\lambda_D^* = 1.0$
Heavy $\lambda_K = 0.2$ $\lambda_Q = 0.3$ $\lambda_D = 0.7$ Note: λ_Q can be calculated based on shear strength at high ductility. See Section 5.3.6.	*Criteria:* • Shear crack widths may exceed 1/8 in., but do not exceed 3/8 in. Higher cracking width is concentrated at one or more cracks. *Typical Appearance:*  Note: *l_p is length of plastic hinge. See Section 5.3.3*	• Replacement or enhancement is required for full restoration of seismic performance. • For <u>partial</u> restoration of performance, inject cracks $\lambda_K^* = 0.5$ $\lambda_Q^* = 0.8$ $\lambda_D^* = 0.8$
Extreme	*Criteria:* • Reinforcement has fractured. *Typical Indications* • Wide shear cracking typically concentrated in a single crack.	• Replacement or enhancement required.

RC1C	COMPONENT DAMAGE CLASSIFICATION GUIDE	System: **Reinforced Concrete**
		Component Type: **Isolated Wall or Stronger Pier**
		Behavior Mode: **Flexure/Web Crushing**

How to distinguish behavior mode:

By observation:
For insignificant-to-moderate levels of damage, indications will be similar to those for RC1A and RC1B. At higher levels of damage, extensive diagonal cracking and spalling of web regions begins to occur.

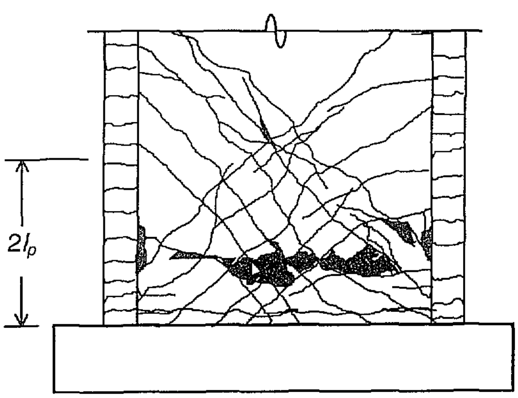
Typically occurs in walls that have sufficient horizontal reinforcement, and that may have heavy vertical (flexural) reinforcement. May be more prevalent in low-rise walls, walls with higher axial loads, and in walls with flanges or heavy boundary elements.

By analysis:
Web crushing strength, calculated for high levels of story drift or ductility, is less than flexural strength.

Foundation rocking strength exceeds moment strength. Boundary ties are sufficient to prevent buckling of longitudinal bars and loss of confinement prior to web-crushing failure. Wall thickness is sufficient to prevent overall buckling prior to web crushing failure. Sliding shear strength is not exceeded.

Refer to Evaluation Procedures for:

- Identifying plastic hinge locations and extent.
- Identifying flexural versus shear cracks.
- Calculation of moment, diagonal tension, web-crushing, sliding-shear, lap splice, and foundation rocking strength.
- Required boundary ties and wall thickness.

Severity	Description of Damage	Performance Restoration Measures
Insignificant	$\mu_\Delta \leq 3$ See **RC1B**	See **RC1B**
Slight	Not Used	
Moderate $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Criteria:* • Shear crack widths do not exceed 1/8 in., and • Flexural crack widths do not exceed 1/4 in., and • Limited spalling (or incipient spalling as identified by sounding) occurs at web or toe regions, or shear cracks exceed 1/16 in., and • No buckled or fractured reinforcement, and • No significant residual displacement.	• Remove and patch spalled and loose concrete. Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Heavy $\lambda_K = 0.2$ $\lambda_Q = 0.3$ $\lambda_D = 0.7$	*Criteria:* • Significant spalling of concrete in web, and • No fractured reinforcement. *Typical Appearance:*  Note: *l_p is length of plastic hinge. See Section 5.3.3*	• Remove and patch all spalled and loose concrete. Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Extreme	*Criteria:* • Heavy spalling and voids in web concrete, or significant residual displacement.	• Replacement or enhancement required.

RC1D COMPONENT DAMAGE CLASSIFICATION GUIDE

System:	**Reinforced Concrete**
Component Type:	**Isolated Wall or Stronger Pier**
Behavior Mode:	**Flexure/Sliding Shear**

How to distinguish behavior mode:

By observation:
For insignificant-to-moderate levels of damage, indications will be similar to those for RC1A. In the plastic hinge zone, flexural cracks join up across the section, which becomes a potential sliding plane. At higher levels of damage, degradation of the concrete and sliding along this crack begin to occur.

Typically occurs in low-rise walls that have sufficient horizontal reinforcement. Sliding may occur at horizontal construction joints. May be more prevalent in walls with lower axial loads, and in walls with flanges or heavy boundary elements. Unlikely to occur if diagonal reinforcement crosses the potential sliding plane.

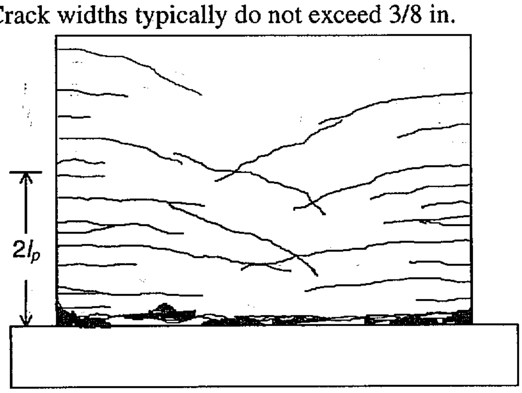
By analysis:
Sliding shear strength is less than shear corresponding to moment strength.

Strength associated with diagonal tension, web crushing, and lap splices — taken for conditions of high ductility — exceeds moment strength. Foundation rocking strength exceeds moment strength. Boundary ties are sufficient to prevent buckling of longitudinal bars and loss of confinement prior to sliding. Wall thickness is sufficient to prevent overall buckling.

Boundary ties are insufficient to prevent bar buckling or provide adequate confinement.

Refer to Evaluation Procedures for:

- Identifying plastic hinge locations and extent.
- Identifying flexural versus shear cracks.
- Calculation of moment, diagonal tension, web-crushing, sliding-shear, lap splice, and foundation rocking strength.
- Required boundary ties and wall thickness.

Severity	Description of Damage	Performance Restoration Measures
Insignificant	See **RC1A**	See **RC1A**
Slight	See **RC1A**	See **RC1A**
Moderate	Not Used	
Heavy $\lambda_K = 0.4$ $\lambda_Q = 0.5$ $\lambda_D = 0.8$	*Criteria:* • Development of a major horizontal flexural crack along the entire wall length, with some degradation of concrete along the crack, indicating that sliding has occurred. Possible small lateral offset at crack. *Typical Appearance:* Crack widths typically do not exceed 3/8 in.  Note: *l_p is length of plastic hinge. See Section 5.3.3*	• Remove and patch all spalled or loose concrete. Inject cracks. $\lambda_K^* = 0.8$ $\lambda_Q^* = 1.0$ $\lambda_D^* = 1.0$
Extreme	*Criteria:* • Significant lateral offset at sliding plane	• Replacement or enhancement required.

RC1E	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Concrete**
		Component Type:	**Isolated Wall or Stronger Pier**
		Behavior Mode:	**Flexure/Boundary Compression**

How to distinguish behavior mode:

By observation:

For insignificant-to-moderate levels of damage, indications will be similar to those for RC1A (although spalling may occur at lower ductility levels). At higher levels of damage, boundary regions in plastic hinge zone begin to sustain spalling and crushing.

Flexure/boundary compression typically occurs in walls that have sufficient horizontal reinforcement and do not have well confined boundary regions. May be more prevalent in walls with a higher M/Vl_w ratio.

Caution: When vertical cracks or spalling at boundary regions is observed, boundary reinforcement should be exposed and inspected for buckling or cracking.

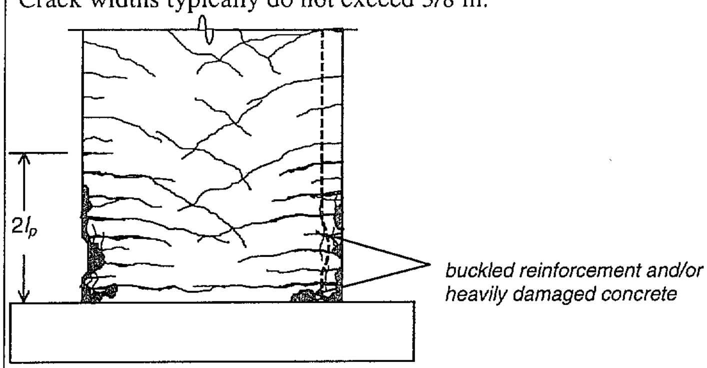
By analysis:

Strength in all other behavior modes, even after possible degradation, is sufficient to ensure that flexural behavior controls. Strength associated with shear, web crushing, sliding shear, and lap splices — taken for conditions of high ductility — exceeds moment strength. Foundation rocking strength exceeds moment strength. Wall thickness is sufficient to prevent overall buckling.

Boundary ties are insufficient to prevent bar buckling or provide adequate confinement.

Refer to Evaluation Procedures for:

- Identifying plastic hinge locations and extent.
- Identifying flexural versus shear cracks.
- Required boundary ties and wall thickness.
- Calculation of moment, diagonal tension, web-crushing, sliding-shear, lap splice, and foundation rocking strength.

Severity	Description of Damage	Performance Restoration Measures
Insignificant	See **RC1A**	See **RC1A**
Slight	See **RC1A**	See **RC1A**
Moderate	See **RC1A**	See **RC1A**
Heavy $\lambda_K = 0.4$ $\lambda_Q = 0.6$ $\lambda_D = 0.7$	*Criteria:* • Spalling or vertical cracking occurs at toe regions in plastic hinge zone, <u>and</u> • Boundary longitudinal reinforcement is buckled <u>or</u> concrete within core of boundary regions (not just cover concrete) is heavily damaged. *Typical Appearance:* Crack widths typically do not exceed 3/8 in.  Note: l_p is length of plastic hinge. See *Section 5.3.3*	• Remove spalled and loose concrete. Remove and replace buckled reinforcement. Provide additional ties around longitudinal bars of the critical boundary region, at the location of the replaced bars. Patch concrete. Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Extreme	See **RC1A**	See **RC1A**

RC2A	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Concrete**
		Component Type:	**Weaker Pier**
		Behavior Mode:	**Ductile Flexural**

How to distinguish behavior mode:

<u>*By observation:*</u>
Wide flexural cracking and spalling should be concentrated in the plastic hinge zone, although minor flexural cracking (width not exceeding 1/8 in.) may extend beyond the plastic hinge zone. Shear cracks may occur but widths should not exceed 1/8 in. Vertical cracks and spalling may occur at the extreme fibers of the plastic hinge region.

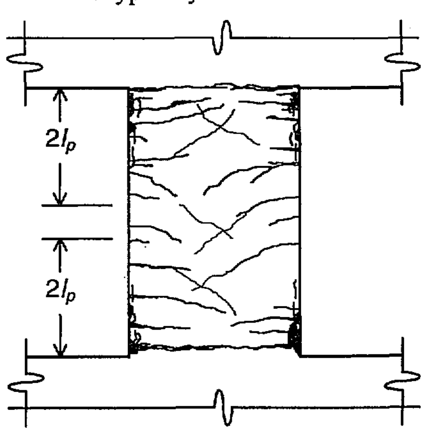
Ductile flexural behavior typically occurs in well-designed, slender wall piers that have sufficient horizontal reinforcement and do not have heavy vertical (flexural) reinforcement.

Note: At low damage levels, damage observations will be similar to those for other behavior modes.

<u>*By analysis:*</u>
Strength in all other behavior modes, even after possible degradation, is sufficient to ensure that flexural behavior controls. Strength associated with shear, web crushing, sliding shear, and lap splices — taken for conditions of high ductility — exceeds moment strength. Foundation rocking strength exceeds moment strength. Boundary ties are sufficient to prevent bar buckling or loss of confinement, and wall thickness is sufficient to prevent overall buckling.

<u>*Refer to Evaluation Procedures for:*</u>

- Identifying plastic hinge locations and extent.
- Identifying flexural versus shear cracks.
- Calculation of moment, diagonal tension, web-crushing, sliding-shear, lap splice, and foundation rocking strength.
- Required boundary ties and wall thickness.

Severity	**Description of Damage**	**Performance Restoration Measures**
Insignificant	See **RC1A**	See **RC1A**
Slight	See **RC1A**	See **RC1A**
Moderate $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Criteria:* • Spalling or vertical cracking (or incipient spalling as identified by sounding) occurs at toe regions in plastic hinge zone, typically limited to the cover concrete, <u>and</u> • No buckled or fractured reinforcement, <u>and</u> • No significant residual displacement. *Typical Appearance:* Crack widths typically do not exceed 1/4 in.  Note: *l_p is length of plastic hinge. See Section 5.3.3*	• Remove and patch spalled and loose concrete. Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Heavy	Not Used	
Extreme	See **RC1A**	See **RC1A**

RC2H	COMPONENT DAMAGE CLASSIFICATION GUIDE	
		System: **Reinforced Concrete**
		Component Type: **Weaker Pier**
		Behavior Mode: **Preemptive Diagonal Tension**

How to distinguish behavior mode:

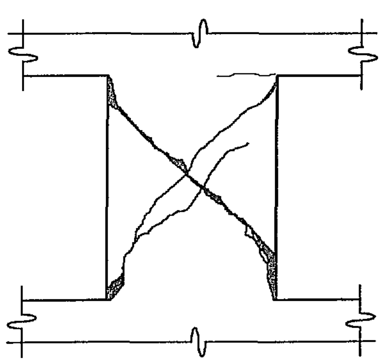
By observation:
For lower levels of damage, indications will be similar to those for other behavior modes, although flexural cracks may not be apparent. Damage quickly becomes heavy when diagonal cracks open up. Because flexural reinforcement never yields, flexural cracks should not have a width greater than 1/8 in.

Preemptive diagonal shear typically occurs in wall piers that have inadequate (or no) horizontal reinforcement, and that may have heavy vertical reinforcement. May be more prevalent in wall piers with low M/Vl_w ratio.

By analysis:
Strength in shear at low ductility is less than the capacity corresponding to moment strength, foundation rocking strength, or lap-splice strength (at low ductility).

Refer to Evaluation Procedures for:

- Identifying flexural versus shear cracks.
- Calculation of moment, shear, lap-splice, and foundation rocking strength.

Severity	Description of Damage		Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* Typical Appearance:	• No shear cracking <u>and</u> • Flexural crack widths do not exceed 1/8 in. Similar to RC2A except no shear cracking and smaller crack widths.	See **RC1A**
Slight	Not Used		
Moderate $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Criteria:* *Typical Appearance:*	• No crack widths exceed 1/8 in. <u>and</u> • No vertical cracking or spalling Similar to insignificant damage except thin shear cracks may be present.	• Inject cracks $\lambda_K^* = 0.8$ $\lambda_Q^* = 1.0$ $\lambda_D^* = 1.0$
Heavy $\lambda_K = 0.2$ $\lambda_Q = 0.3$ $\lambda_D = 0.7$ Note: λ_Q can be calculated based on shear strength at high ductility See Section 5.3.6	*Criteria:* *Typical Appearance:*	• Shear crack widths exceed 1/8 in., but do not exceed 3/8 in. Cracking becomes concentrated at one or more cracks. 	• Replacement or enhancement is required for full restoration of seismic performance. • For <u>partial</u> restoration of performance, Inject cracks. $\lambda_K^* = 0.5$ $\lambda_Q^* = 0.8$ $\lambda_D^* = 0.8$
Extreme	*Criteria:* *Typical Indications*	• Reinforcement has fractured. • Wide shear cracking typically concentrated in a single crack.	• Replacement or enhancement required

RC3B	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Concrete**
		Component Type:	**Coupling Beam**
		Behavior Mode:	**Flexure/Diagonal Tension**

How to distinguish behavior mode:

By observation:

For insignificant-to-moderate levels of damage, indications will be similar to those for RC1A, although shear cracking may begin at lower ductility levels. At higher levels of damage, one or more wide shear cracks begin to form.

Flexure/Diagonal tension typically occurs in coupling beams that have inadequate stirrup reinforcement and that may have heavy horizontal (flexural) reinforcement. More prevalent in deeper beams than in shallower beams, but depending on the reinforcement, can occur over a wide range of aspect ratios.

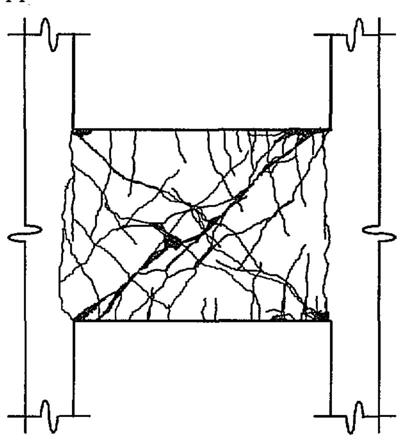
By analysis:

Shear strength calculated for conditions of low ductility exceeds flexural capacity, but shear strength calculated for conditions of high ductility is less than the flexural capacity.

Web crushing strength and sliding shear strength are not exceeded.

Refer to Evaluation Procedures for:

- Identifying plastic hinge locations and extent.
- Identifying flexural versus shear cracks.
- Calculation of moment, diagonal tension, web-crushing, sliding-shear, and lap splice strength.
- Required boundary ties and wall thicknesses.

Severity	Description of Damage	Performance Restoration Measures
Insignificant	See **RC1B**	See **RC1B**
Slight	Not Used	
Moderate	See **RC1B**	See **RC1B**
Heavy $\lambda_K = 0.2$ $\lambda_Q = 0.3$ $\lambda_D = 0.7$ Note: λ_Q can be calculated based on shear strength at high ductility See Section 5.3.6	*Criteria:* • Shear crack widths may exceed 1/8 in., but do not exceed 3/8 in. Higher width cracking is concentrated at one or more cracks. *Typical Appearance:* 	• Replacement or enhancement is required for full restoration of seismic performance. • For partial restoration of performance, Inject cracks. $\lambda_K* = 0.5$ $\lambda_Q* = 0.8$ $\lambda_D* = 0.8$
Extreme	See **RC1B**	See **RC1B**

RC3D	COMPONENT DAMAGE CLASSIFICATION GUIDE		
		System:	**Reinforced Concrete**
		Component Type:	**Coupling Beam**
		Behavior Mode:	**Flexure/Sliding Shear**

How to distinguish behavior mode:

By observation:

For insignificant-to-moderate levels of damage, indications will be similar to those for RC1A. Vertical flexural cracks join up across one or both ends of the section, which become a potential sliding plane. At higher levels of damage, degradation of the concrete and sliding along the critical crack begin to occur.

This behavior typically occurs in coupling beams that do not have diagonal reinforcement, but have sufficient stirrups to prevent diagonal tension failures.

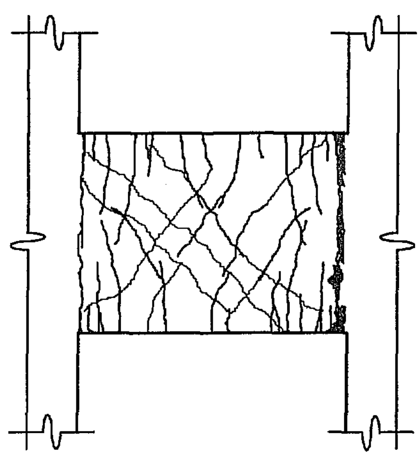
By analysis:

Sliding shear strength is less than shear corresponding to moment strength.

Strength associated with diagonal tension, web crushing, and lap splices for conditions of high ductility exceeds moment strength.

Refer to Evaluation Procedures for:

- Identifying plastic hinge locations and extent.
- Identifying flexural versus shear cracks.
- Calculation of moment, diagonal tension, web-crushing, sliding-shear, and lap splice strength.
- Required boundary ties and wall thickness.

Severity	Description of Damage	Performance Restoration Measures
Insignificant	See **RC1D**	See **RC1D**
Slight	See **RC1D**	See **RC1D**
Moderate	Not Used	
Heavy $\lambda_K = 0.2$ $\lambda_Q = 0.3$ $\lambda_D = 0.7$	*Criteria:* • Development of a major vertical flexural crack along the entire beam depth, with some degradation of concrete along the crack, indicating that sliding has occurred. Possible small lateral offset at crack. *Typical Appearance:* Crack widths typically do not exceed 3/8 in. 	• Remove and patch all spalled or loose concrete. Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Extreme	See **RC1D**	See **RC1D**

6: Reinforced Masonry

6.1 Introduction and Background

This section provides material relating to reinforced masonry (RM) construction and includes the Component Damage Classification Guides (Component Guides) in Section 6.5. Reinforced masonry component types and behavior modes are defined and discussed in Section 6.2. The overall damage evaluation procedure uses conventional material properties as a starting point. Section 6.3 provides supplemental information on strength and deformation properties for evaluating reinforced masonry components. Typical hysteretic behavior for reinforced masonry components and the interpretation of cracking are discussed in FEMA 307. The information presented on reinforced masonry components has been generated from a review of available empirical and theoretical data listed in the reference section and the annotated tabular bibliography in FEMA 307. These provide the user with further detailed resources on reinforced masonry component behavior.

Unreinforced masonry components (URM) are covered in Chapter 7 of this document. The distinction between reinforced and unreinforced masonry can sometimes be an issue. In those cases, masonry with less that 25 percent of the recommended minimum reinforcement specified in FEMA 273 should be considered unreinforced.

The most effective first step in identifying reinforced masonry components and their likely behavior modes is to place the structure in the context of the history of local construction practices, and to determine the type and amount, if any, of reinforcement used. There are examples of the use of iron to reinforce brick masonry construction in the 19th century; however, the widespread use of modern reinforced masonry did not begin until the 1930s. The use of reinforced masonry building systems was accelerated on the west coast following the 1933 Long Beach earthquake when the use of unreinforced masonry for new buildings in California was prohibited, so there is a distinct difference in building types in California before and after 1933. Reinforced masonry construction technology also developed in the east, although unreinforced masonry structures may still be built in some areas. The use of Portland cement mortars increased steadily from the beginning of the 20th century, as did the strength and quality of fired clay masonry. FEMA 274, Chapter 7, includes additional information on the history of masonry construction in the United States, as does the Brick Institute of America "Technical Notes on Brick Construction, No. 17." (BIA, 1988)

A wide variety of construction systems may be classified as reinforced masonry. The most common are:

- Fully-grouted hollow concrete block
- Partially-grouted hollow concrete block
- Fully-grouted hollow clay brick
- Partially-grouted hollow clay brick
- Grouted-cavity wall masonry (two wythes of clay brick or hollow units with a reinforced, grouted cavity)

Most of these are addressed in this section; however, the quantity and quality of experimental data available for each type varies considerably.

The last twenty-five years have seen a dramatic increase in masonry research over that in prior years, as evidenced by the proceedings of the International Brick/Block Masonry Conferences (1969 - present), The North American Masonry Conferences (1976 - present), and the Canadian Masonry Symposia (1976 - present). Much of this work has been directed toward measuring strength and serviceability characteristics under gravity or wind loading or toward development of working-stress design methods. Since the early eighties, a growing number of studies have addressed the strength and deformation characteristics of reinforced masonry components under cyclic (simulated seismic) loading. Notable early studies include those at the University of California, San Diego (e.g., Hegemier et al., 1978), University of California, Berkeley (e.g., Hidalgo et al., 1978 and 1979), and the University of Canterbury at Christchurch, New Zealand (e.g., Priestley and Elder, 1982). In 1985, the Technical Coordinating Committee for Masonry Research (TCCMAR) organized the U.S.-Japan Coordinated Program for Masonry Building Research. The majority of experimental data available today for the complete load-displacement response of reinforced masonry under fully-reversed cyclic loads (static and dynamic) were generated in this program (Noland, 1990). The U.S.-Japan Coordinated Program for Masonry Building Research (often referred to as the

"TCCMAR program") included experimental and analytical studies on the seismic response of reinforced masonry materials, components, seismic structural elements, and complete building systems. Documentation of the data was thorough, and coordination of materials and methods between different research institutions was carefully controlled. Noland (1990) provides a complete list of experimental studies and associated publications.

Despite the variety of reinforced masonry systems in use, most of the TCCMAR research and earlier cyclic-loading studies were conducted with fully-grouted, hollow concrete block masonry. Most of the Component Damage Classification Guides for reinforced masonry in this document therefore apply most directly to fully-grouted concrete block masonry. A series of coordinated studies (Atkinson and Kingsley, 1985; Young and Brown, 1988; Hamid et al., 1989; Shing et al., 1991; Blondet and Mayes, 1991; Agbabian et al., 1989) have shown that the behavior characteristics of hollow concrete and hollow clay masonry in compression, in-plane flexure, and out-of-plane flexure are quite similar in terms of ductility and energy-dissipation characteristics, although clay masonry is generally of significantly higher strength. Clay masonry is also more likely to exhibit brittle characteristics and separation of faceshells from grout, whereas concrete masonry with well-designed grout can behave more homogeneously. For the purposes of this document, the behavior of fully-grouted hollow clay and hollow concrete masonry is assumed to be identical.

Relatively little work has been conducted on the seismic response of partially-grouted masonry. An extensive study of partially-grouted shear walls was conducted by NIST (Fattal, 1993), but the emphasis in reported results was on shear strength only. Schultz (1996) reports that in-plane response of partially-grouted walls with light horizontal reinforcement is characterized by vertical cracking at the junction of grouted and ungrouted vertical cells, propagating between horizontally grouted cells. Load degradation is associated with widening of the vertical cracks to 0.25" and greater. Masonry pier tests conducted at the University of California, Berkeley (Hidalgo et al., 1978; Chen et al., 1978; and Hidalgo et al., 1979) included several partially-grouted specimens. Damage patterns for these specimens were not so different from fully-grouted specimens, and strength was only mildly affected by partial grouting. However, deformation capacity was dramatically decreased relative to identical walls with full grouting.

Seismic response of grouted brick-cavity wall masonry has also received relatively little attention. The masonry pier tests conducted at UC Berkeley (Hidalgo et al., 1978; Chen et al., 1978; and Hidalgo et al., 1979) included 18 tests on two-wythe, grouted clay brick masonry. Failure modes were similar to those for hollow clay masonry, but tended to be more brittle, involving the development of vertical splitting cracks between the brick wythes and the grout. Horizontal reinforcement had little or no effect on the behavior of grouted brick-cavity walls failing in shear. This can be attributed to the rapid failure and delamination of the brick wythes, leaving a narrow and unstable grout-reinforced core that was incapable of developing a stable flexural compression zone.

Component Damage Classification guides for reinforced masonry reflect the availability of experimental data for each of the reinforced masonry systems. Reinforced masonry systems that are not well represented by experimental tests are not included in the guides.

6.2 Reinforced Masonry Component Types and Behavior Modes

6.2.1 Component Types

Component types for reinforced masonry are conceptually very similar to those for reinforced concrete (see Chapter 5). Table 6-1 lists four common reinforced masonry component types. Note that components are distinguished in terms of both geometric characteristics and behavior modes.

Each component defined in Table 6-1 may suffer from different types of damage, acting either in a pure behavior mode such as flexure, or, more likely, in a mixed mode such as flexure degrading to shear or sliding-shear failure. Table 6-2 outlines the likelihood of different behavior modes occurring in components RM1 through RM4, and references the relevant Component Guides in Section 6.5.

Table 6-3 outlines the manner in which the strength and deformation capacity of each behavior mode may be evaluated. A detailed description of each entry in Table 6-3 is given in Section 6.3. Additional example hysteresis curves are provided in FEMA 307, Section 3.

Table 6-1 ***Component Types for Reinforced Masonry***

Component Type		Description
RM1	Stronger pier	Examples are cantilever walls that ultimately are controlled by capacity at their base (e.g., flexural plastic hinge, shear failure, rocking) and story-height wall piers that are stronger than spandrels that frame into them. Wall components may be rectangular (planar) or may include out-of-plane components (flanges) that can have a significant effect on the response.
RM2	Weaker pier	Wall piers controlled by shear failure (more likely) or flexural hinging at the top and bottom (less likely). Wall components may be rectangular (and planar) or may include out-of-plane components (flanges) that can have a significant effect on the response.
RM3	Weaker spandrel or coupling beam	Masonry beams that are weaker than the wall piers into which they frame. These are often controlled by shear capacity and less frequently by flexure.
RM4	Stronger spandrel or coupling beam	Masonry beams that are stronger than the wall piers into which they frame.

6.2.2 Behavior Modes with High Ductility

Reinforced masonry structural components with relatively high ductility exhibit some of the following common attributes:

- Wall piers with aspect ratios (height / length) of two or greater or spandrels with span to depth ratios of four or greater.
- Moderate levels of axial load ($P/A_g < 0.10f_{me}$). High axial loads decrease ductility by increasing the strain in the flexural compression zone, resulting in crushing at lower curvatures than in lightly-loaded walls. Walls with very low levels of axial load may be limited by sliding shear capacity.
- Relatively large flexural demand compared to corresponding shear. An example is a wall with flexible or weak spandrels. The lack of significant intermediate rotational restraint on the wall leads to cantilever behavior with relatively high M/V ratios as compared to frame behavior.
- Very small, or no, tension flange. Development of reinforcement in tension flanges can result in over-reinforced sections, dramatically limiting ductility.
- Uniformly distributed reinforcement.
- Sufficient shear reinforcement to ensure flexural response

The initial expected strength of a ductile reinforced masonry component in flexure is given by the in-plane moment strength as defined in Section 7.4.4 of FEMA 273. Flanges, particularly on the tension side, should be included as part of the critical section according to the limits set in FEMA 273. It is also important to consider the effects of axial load, and to consider all reinforcement in the wall as effective. The theoretical basis for calculating flexural strength of reinforced masonry walls follows the well-established principles of ultimate strength design for reinforced concrete, and there is sufficient experimental data to support its use for masonry. For additional discussion, see Priestley and Elder (1982), Shing et al. (1991), Kingsley et al. (1994), and Seible et al. (1994b).

The displacement capacity of a ductile flexural wall can be determined with reasonable accuracy by idealizing it as a cantilever beam and calculating the flexural and shear deformations. Displacements following cracking, but prior to significant yielding, may be approximated using an effective cracked stiffness (Priestley and Hart, 1989). After yielding, the wall can be idealized as having an equivalent plastic-hinge zone at the base, and displacement can be calculated using the methods presented in Paulay and Priestley (1992).

With increasing distance from the plastic-hinge zone, the contribution of shear deformations to displacements is less significant, and a pure flexural model is sufficient. Seible et al. (1995) showed that at the maximum displacement in a five-story, full-scale reinforced masonry building, the shear deformation component of lateral displacement was as high as 50 percent at the first story, and less than 10 percent at the fifth floor. Priestley and Elder (1982), Leiva and Klingner (1991), Shing et al. (1990a, b), and Kingsley et al. (1994) provide additional experimental evidence to support the calculation of displacements in ductile flexural walls. The probable displacement capacity of ductile flexural walls should be at least four times the yield displacement, or one percent of building drift.

Table 6-2 Likelihood of Earthquake Damage to Reinforced Masonry Components According to Component and Behavior Mode.

Ductility		Behavior Mode	Wall Component Type			
			RM1 Stronger Pier	**RM2 Weaker Pier**	**RM3 Weaker Spandrel**	**RM4 Stronger Spandrel**
High ductility	A	Flexure	Common See Guide RM1A	Unlikely	Common See Guide RM3A	N/A
		Foundation rocking	May occur, but not considered See FEMA 273 or ATC-40	May occur, but not considered See FEMA 273 or ATC-40	N/A	N/A
Moderate ductility	B	Flexure / Diagonal shear	Common See Guide RM1B	Common See Guide RM2B	May occur Similar to Guide RM3A	N/A
	C	Flexure / Sliding shear	May occur See Guide RM1C	May occur Similar to Guide RM1C	Unlikely	N/A
	D	Flexure / Out-of-plane instability	May occur following large displacement cycles See Guide RM1D	Unlikely	Unlikely	N/A
	E	Flexure / Lap splice slip	May occur See Guide RM1E	Unlikely	May occur	N/A
	F	Pier rocking	May occur Similar to Guide RM1E	May occur Similar to Guide RM1E	N/A	N/A
Little or no ductility	G	Preemptive diagonal shear	Common Similar to Guide RM2G	Common See Guide RM2G	Common See Guide RM3G	N/A
	H	Preemptive sliding shear	May occur in poorly detailed wall Similar to Guide RM1C	May occur in poorly detailed wall Similar to Guide RM1C	N/A	N/A

Notes:
- Shaded areas of the table with notation "*See Guide...*" indicate behavior modes for which a specific Component Guide is provided in Section 6.5. The notation "*Similar to Guide...*" indicates that the behavior mode can be assessed by using the guide for a different, but similar component type or behavior mode.
- *Common* indicates that the behavior mode has been evident in postearthquake field observations and/or that experimental evidence supports a high likelihood of occurrence.
- *May occur* indicates that a behavior mode has a theoretical or experimental basis, but that it has not been frequently reported in postearthquake field observations.
- *Unlikely* indicates that the behavior mode has not been observed in either the field or the laboratory.
- N/A indicates that the failure mode cannot occur for that component.

Table 6-3 ***Behavior Modes for Reinforced Masonry Components (Note: Hysteresis Curves from Shing et al., 1991)***

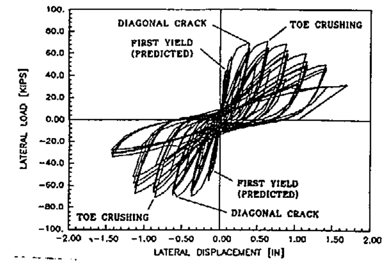
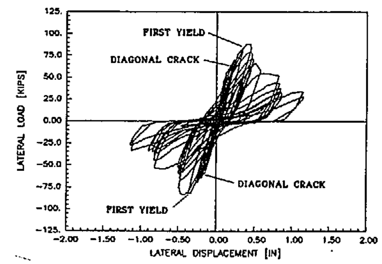
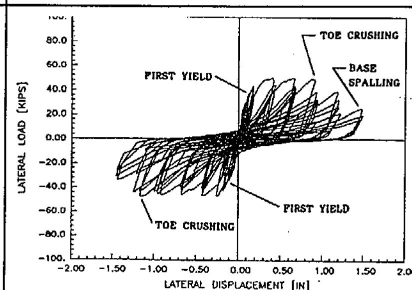
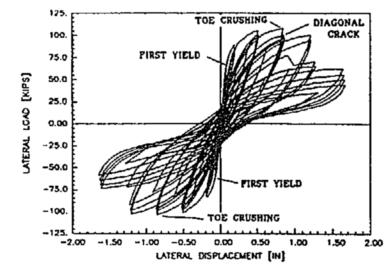
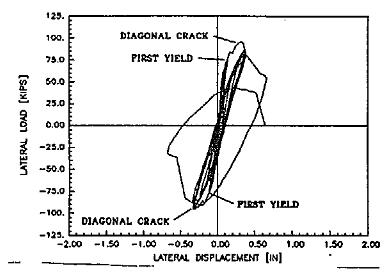
Behavior Mode	Approach to calculate strength (use expected material values)	Approach to estimate displacement capacity	Ductility Category	Example hysteresis loop shape
A. Ductile Flexure	The expected strength under in-plane forces is limited by the development of the expected moment strength, M_e. This is calculated considering all distributed steel, axial loads, and the development of tension flanges, if present. Note that the maximum possible strength is greater than the expected strength, which may influence the governing mode. See Section 6.3.2a	The displacement capacity is limited by the maximum curvature attained within the effective plastic-hinge zone. Classical moment-curvature analysis may be used and related to displacement with some empirical calibration. See Sections 6.3.2b and 6.3.2c	High ductility capacity See Section 6.2.2	
B. Flexure / Shear	The initial expected strength is governed by M_e, as calculated for the ductile flexural mode. Strength degrades as the masonry component of the shear strength, V_m, degrades, with residual strength governed by the reinforcement component V_s. See Sections 6.3.2a and 6.3.3a	Displacement capacity at the initial expected strength may be estimated as the intersection of the flexural load-displacement curve with the degrading shear-strength envelope. See Section 6.3.3b	Moderate ductility capacity See Section 6.2.3	
C. Flexure / Sliding shear	The initial expected strength is governed by M_e, as calculated for the ductile flexural mode. Stiffness and strength degrade as sliding-shear mode develops, and hysteresis becomes pinched. The initial strength may be maintained, but only at large displacements. See Sections 6.3.2a and 6.3.4	Displacements due to sliding may be large. Displacement capacity for ductile flexural behavior may provide reasonable estimate. See Section 6.3.4b	Moderate-to-high ductility capacity See Section 6.2.2 and 6.2.3	

Table 6-3 ***Behavior Modes for Reinforced Masonry Components (Note: Hysteresis Curves from Shing et al., 1991) (continued)***

D. Flexure / Out-of-plane stability	The initial expected strength is governed by M_e, as calculated for the ductile flexural mode. Following instability failure, strength drops rapidly. See Sections 6.3.2a and 6.3.5	Displacement capacity is limited by the slenderness of the wall with respect to the height and/or the length. See Section 6.3.5	Moderate-to-high ductility capacity See Section 6.2.2 and 6.2.3	
E. Flexure / Lap splice slip	The initial expected strength is governed by M_e, as calculated for the ductile flexural mode. With failure of lap splices, strength degrades to rocking mode, limited by crushing of wall toes. See Sections 6.3.2a and 6.3.6	Displacement capacity at expected strength limited by lap-splice slip. See Section 6.3.6	Moderate ductility capacity See Section 6.2.3	
G. Preemptive diagonal shear	The expected strength is reached before the development of the expected moment capacity and is governed by shear strength, V_e. See Section 6.3.3	No inelastic capacity.	No ductility capacity See Section 6.2.4	
H. Preemptive sliding shear	The expected strength is reached prior to the development of the expected moment capacity, and is governed by the sliding shear strength, V_{se}. See Section 6.3.4	Little displacement capacity, limited by crushing of bottom course of masonry and/or buckling of vertical reinforcement.	Little ductility capacity See Section 6.2.4	

Damage in flexural walls is likely to include both horizontal and diagonal cracks of small size concentrated in the plastic-hinge region. Diagonal cracks typically propagate from horizontal, flexural cracks, and therefore have similar, regular spacing. At the large displacements, crushing may occur at the wall toes.

Another relatively ductile behavior mode is foundation rocking. This can occur if the rocking capacity of the foundation is less than the strength of the wall component it supports. Foundation components are covered in FEMA 273 and ATC-40.

6.2.3 Behavior Modes with Moderate Ductility

Moderately-ductile components initially behave similarly to highly-ductile components, but their ultimate displacement capacity is limited by the influence of less-ductile modes such as sliding or diagonal shear. The response of moderately-ductile components is difficult to predict analytically due to the complex interaction of moment, shear, and axial load, and less difficult to recognize in a damaged component. The majority of experimental data for reinforced masonry components falls into this moderately ductile category, For some examples, refer to Shing et al. (1991).

The initial strength is governed by the flexural capacity; however, the initial strength cannot be maintained at high ductility levels. Displacement capacity for moderate-ductility modes is difficult to calculate. Research is currently underway to improve the ability to predict displacements associated with diagonal shear modes of behavior, but there are currently no established guidelines, with the exception of the semi-empirical recommendations in FEMA 273.

At low levels of response, damage in moderately ductile components resembles that for ductile components, consisting primarily of horizontal flexural and diagonal shear cracks. The component response at larger displacements depends on the governing behavior mode, as described in the following paragraphs.

a. Flexure / Diagonal shear

Diagonal shear response is characterized by the growth of diagonal cracks accompanied by degrading strength. Eventually, cracks cross the entire length of the wall, and the residual strength of the wall is that provided by the horizontal reinforcement alone. Extensive experimental evidence is available to document this behavior mode, including Shing et al. (1991), Hidalgo et al. (1978), and Chen et al.(1978).

b. Flexure / Sliding shear

Walls may be susceptible to sliding-shear mechanisms when axial load levels are low, vertical reinforcement ratios are low, or when very large ductilities are achieved and the shear friction mechanism degrades. At low displacements, sliding may be observed as a simple lateral offset in a wall. At very large displacements, localized crushing of the bottom course of masonry can result, and vertical reinforcement can experience large lateral offsets.

c. Flexure / Out-of-plane instability

At high ductility levels, the flexural compression zone of slender walls may be susceptible to instability after the development of large tensile strains during previous cycles. This type of failure has been observed in laboratory tests of well-detailed, highly-ductile flexural walls, (see Paulay and Priestley, 1993) but it has not been noted in the field. Out-of-plane instability would not be expected in walls with flanges at the end of the wall, or in very thick walls.

d. Flexure / Lap splice slip

If starter bars with insufficient development length are located at the base of structural walls, overturning forces can result in bond degradation and eventual rocking of the wall on the foundation. Local damage may appear first as vertical cracks at the location of the lap splices, and eventually crushing at the wall toes. (Priestley et al., 1978).

6.2.4 Behavior Modes with Low Ductility

General characteristics of reinforced masonry components exhibiting low-ductility behavior include:

- Wall piers with aspect ratios (height / length) of less than 0.8 and spandrels with span-to-depth ratios of less than two
- High levels of axial load ($P_u/f_{me} A_g > 0.15$)
- Large tension flanges connected continuously to the component

- Large amounts of flexural reinforcement at component edges
- Light shear reinforcement relative to flexural reinforcement

Experimental research on non-ductile walls under cyclic load histories includes Shing et al. (1991), Hidalgo et al. (1978), Chen et al. (1978), and Hidalgo et al. (1979).

Flexural capacity does not govern the nonductile modes of failure. Strength is defined by the diagonal shear strength or the horizontal sliding strength. In the first case, diagonal shear failure causes the lateral capacity of the wall to be immediately reduced to the capacity of the horizontal reinforcement alone. In the latter case, preemptive sliding of the wall does not allow the development of the full flexural capacity, resulting in large displacements with little capacity to dissipate hysteretic energy. It should be noted that bed-joint sliding in URM components may be considered as relatively ductile behavior. Calculation of the shear strength of masonry structural walls is addressed by Leiva and Klingner (1991), Shing et al. (1991), and Anderson and Priestley (1992).

Components that experience preemptive, force-controlled failures cannot be considered to have dependable inelastic displacement capacity.

Diagonal shear failure can occur with little or no early indication of incipient failure. Damage is characterized by one or two dominant diagonal cracks of large width. Damage may ultimately include crushing and spalling in the central portion of the wall. Walls that fail in sliding shear may have very little cracking or damage outside the sliding joint. Ultimately, crushing and spalling of the base course of masonry units can occur.

6.3 Reinforced Masonry Evaluation Procedures

This section provides the basis for calculating the strength and deformation capacities of reinforced masonry components both before and after a damaging earthquake. Subsections are organized according to behavior modes.

6.3.1 Material Properties

The procedures for evaluating strength and deformation capacities presuppose the knowledge of component characteristics, including dimensions, amounts and location of reinforcement and the material properties. Methodologies for structural investigation and the evaluation of these parameters are given in Chapter 3. Additional guidelines for estimating masonry material properties are given in FEMA 273, Section 7.3.2.

If no information from testing is available, initial assumptions for expected material properties as given in FEMA 273 and summarized in Table 6-4 may be assumed.

a. Masonry

Table 6-4 Initial Expected Clay or Concrete Masonry Properties

Condition	Expected Strength f_{me} (psi)	Elastic Modulus (psi)	Friction Coefficient
Good	900	$550 f_{me}$	0.7
Fair	600	$550 f_{me}$	0.7
Poor	300	$550 f_{me}$	0.7

If observed failure modes are not consistent with the initial material strength values given above, actual expected values may be substantially greater (more than three times the table values).

b. Reinforcing steel

Recommendations in FEMA 273 Section 6.4.2.2. are adopted here for yield strength of reinforcement. In the absence of applicable test data, the expected strength of yielding reinforcement, f_{ye}, is assumed to be equal to 1.25 times the nominal yield stress. A range of reinforcement strength values between 1.1 and 1.4 times the nominal yield strength can also be considered in the evaluation procedures.

6.3.2 Flexure

a. Strength

The in-plane flexural capacity of a reinforced masonry wall with distributed reinforcement may be calculated based on the well-established principles of ultimate strength design, as stated in FEMA 273, Section

7.4.4.2.A. It is convenient to express this moment strength in terms of contributions from the masonry, the reinforcement, and the axial load independently, including all distributed reinforcement (Paulay and Priestley, 1992), as shown in Equation 6-1.

$$M_e = C_m\left(c - \frac{a}{2}\right) + \sum_{i=1}^{n} \left| f_{ye} A_{si} (c - x_i) \right| + P_u \left(\frac{l_w}{2} - c \right) \quad (6\text{-}1)$$

Where:

- M_e = expected moment capacity of a masonry section
- C_m = compression force in the masonry
- f_{ye} = expected reinforcement yield strength
- A_{si} = area of reinforcing bar i
- x_i = location of reinforcing bar i
- c = depth to the neutral axis
- a = depth of the equivalent stress block
- P_u = wall axial load
- l_w = length of the wall

Note that all bars are considered to participate, and it is assumed for the purpose of calculating the moment that all bars are yielding. This expression arbitrarily sums moments about the neutral axis of the section. In some cases, it may be more convenient to use a different location. For example, the axial component often passes through the centroid of the section and can be eliminated from the summation of moments at the centroid; however, its effect on moment capacity must be included by proper calculation of C_m.

The expression for moment strength is valid for masonry walls with reinforcement concentrated in the wall boundaries. Walls with concentrated reinforcement, particularly when larger bar sizes are used, are vulnerable to grout flaws in the wall toes. These walls are more likely to develop lap-splice slip or sliding-shear behavior modes.

b. Deformation

A ductile flexural component can be idealized as a cantilever element with a zone of concentrated plastic rotation at the base (the equivalent plastic hinge). Paulay and Priestley (1992) provide a simple model for calculating displacements. At first yield, the displacement at the level of the horizontal force resultant (i.e., the effective height), is

$$\Delta_y = \frac{\phi_y h_e^2}{3} \quad (6\text{-}2)$$

Where:

- ϕ_y = yield curvature of a masonry section
- h_e = effective height of the wall

The maximum displacement capacity at the effective height is:

$$\Delta_p = (\phi_m - \phi_y) l_p (h_e - 0.5 l_p) \quad (6\text{-}3)$$

Where:

- ϕ_m = maximum plastic curvature of a masonry section
- l_p = effective plastic-hinge length (see Section 6.3.2c)

The displacement ductility is:

$$\mu_\Delta = \frac{\Delta_y + \Delta_p}{\Delta_y} = 1 + \frac{\Delta_p}{\Delta_y} \quad (6\text{-}4)$$

c. Plastic-Hinge Length

The concept of a plastic hinge in masonry is adopted as a computational convenience to describe in simple terms the complex distribution of cracks and the localized inelastic deformations in reinforcement. While there is no plastic hinge at a point *per se*, there is a zone over which the curvature may be expected to exceed the yield curvature at large displacements. The following expression for plastic-hinge length has been shown to agree reasonably well with experimental results for reinforced masonry walls (Paulay and Priestley, 1993), and is given in FEMA 274 Section C7.4.4.3A:

$$l_p = 0.2 l_w + 0.04 h_e \quad (6\text{-}5)$$

Where:

- l_w = length of the wall
- h_e = height to the resultant of the lateral force = M/V

See also Shing et al. (1990a, b).

For the purposes of this document, the plastic-hinge length is useful for calculating ultimate displacements in flexural walls, and for identifying the zone over which to expect degradation in shear strength with increasing ductility.

d. Flanges

When a flexural wall includes flanges, there is a potential to develop the flange reinforcement in flexural tension, thus increasing the flexural strength, and potentially decreasing the ductility capacity. In reinforced masonry, a flange can only be engaged when reinforcement and grout are continuous around the wall intersections. When calculating the flexural capacity of a flanged wall, FEMA 273, Section 7.4.4.2.C recommends that an effective flange width equal to 3/4 of the effective wall height ($h_e = M/V$) should be assumed, (He and Priestley, 1992; Seible et al., 1994b).

Damage patterns in fully-engaged flanges appear as horizontal cracks, and possibly as the continuation of diagonal cracks from the in-plane wall.

e. Coupling

Coupling between wall pier and spandrel components causes cyclic axial loads in the piers generated by shear in the spandrels. When the cyclic axial force is compressive, the pier strength is increased, and the ductility decreased. Similarly, when the axial force is tensile, the strength is decreased and the ductility is increased. For walls with relatively little gravity load, the tension force due to coupling can be sufficient to place the wall in a state of net tension. For the purposes of damage classification, the coupling-induced axial loads are to be considered when identifying the governing behavior mode.

Experimental data for reinforced masonry coupled walls is given in Seible et al. (1991), Paulay and Priestley (1992), and Merryman et al. (1990).

6.3.3 Shear

a. Strength

The in-plane shear capacity of an undamaged structural wall may be calculated using the recommended procedure in FEMA 273, Section 7.4.4.2.B, which may be expressed as the sum of three components corresponding to the contributions of masonry, reinforcement, and axial load, respectively:

$$V_e = V_m + V_s + V_p \qquad (6\text{-}6)$$

where:

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vl_w}\right)\right] A_g \sqrt{f_{me}} \qquad (6\text{-}7)$$

$$V_s = 0.5\left(\frac{A_v}{s}\right) f_{ye} d_{vs} \qquad (6\text{-}8)$$

$$V_p = 0.25 P_u \qquad (6\text{-}9)$$

In FEMA 273, the V_p component is included as a part of the V_m component; they are expressed separately here to facilitate the following discussion of degrading shear strength. Note that in FEMA 273, the total shear strength V_e is limited to:

$$V_e = 6A_g\sqrt{f_{me}} \text{ for walls with } M/V_d < 0.25 \qquad (6\text{-}10)$$

$$V_e = 4A_g\sqrt{f_{me}} \text{ for walls with } M/V_d \geq 1.00 \qquad (6\text{-}11)$$

Equations 6-10 and 6-11 describe the maximum shear strength of an initially undamaged wall. As a flexural wall undergoes cyclic displacements, horizontal cracks initiate on the tension side of the wall and propagate towards the neutral axis, and diagonal cracks initiate near the center of the wall and propagate outward. As cracks open, horizontal reinforcement is engaged, and the mechanism of shear resistance in the masonry changes. The bulk of the masonry shear is transferred through the flexural compression zone – where the local shear strength is enhanced by the increasing compression stresses – and the remainder is transferred through aggregate interlock across the cracks. This mechanism degrades as both flexural and shear cracks open wider, until the capacity of the wall is reduced to nearly that of the horizontal reinforcement alone. Priestley, et al. (1994) have developed a model that captures this response for concrete columns, but such a relationship has not yet been developed and verified for masonry walls. Because it is nonconservative to ignore the degrading strength, however, the following relationship may serve for masonry in the area of the plastic-hinge zone until an improved model can be developed:

$$V_m = k\left[4.0 - 1.75\left(\frac{M}{Vl_w}\right)\right]A_g\sqrt{f_{me}} \qquad (6\text{-}12)$$

where k=1 for displacement ductility values less than 1.5, and reduces linearly to a value of 0.1 at a displacement ductility of 4, and further to 0.0 at a displacement ductility of 8.

b. Deformation

Deformation mechanisms of walls with a predominant shear mode behavior cannot be quantified as simply as those of walls with flexural behavior modes, particularly after significant diagonal cracking and after yielding of reinforcement. As an approximation, the flexural force-displacement relationship can be developed, as described in Section 6.3.2, and the degrading shear strength relationship in Equation 6-12 above may be used to identify the displacement at which shear modes of behavior begin to dominate the response.

6.3.4 Sliding

a. Strength

The ability of a structural masonry wall to resist sliding shear may be described in terms of shear friction across a crack or construction joint, as described in ACI 318-95, Chapter 11. This friction may be visualized as having two components, (Paulay and Priestley 1992), the first due to the friction associated with the axial load on the wall, and the second due to the friction associated with the clamping force provided by the vertical reinforcement across the sliding plane, thus:

$$V_{se} = \mu P_u + \mu A_{vf} f_{ye} \qquad (6\text{-}13)$$

Where:

P_u = wall axial load

A_{vf} = area of reinforcement crossing perpendicular to the sliding plane

f_{ye} = expected yield strength of reinforcement

μ = coefficient of friction at the sliding plane

Values for the coefficient of friction may be determined using the recommendations of ACI 318-95, Section 11.7.4.3. Atkinson et al. (1988) determined that for mortared brick masonry joints, a value of 0.7 represents an average expected value.

Uniformly distributed reinforcement is more effective in resisting sliding shear than is reinforcement concentrated at the ends of the wall. Distributed reinforcement leads to a larger flexural compression zone than does concentrated reinforcement, thus enhancing shear transfer across the plane. Distributed reinforcement is also located closer to the rough surfaces that generate the shear friction forces.

Wall components that are classified as RM1 may be particularly vulnerable to sliding-shear behavior, or, more specifically, flexural response that degrades to sliding-shear response. The reason for this vulnerability is that, at the large curvature ductilities developed in the plastic-hinge zones of flexural walls, horizontal cracks open wide and cause the reinforcement across the sliding plane to yield. As the cracks open, the potential to develop the shear friction mechanism degrades, leaving only the comparatively flexible dowel-action mechanism of the reinforcing bars (Paulay and Priestley, 1992). Under cyclic reversals at large curvature ductility, it is possible to open a horizontal crack across the entire length of a wall. Sliding behavior is well documented for reinforced masonry walls (Shing et al., 1991), particularly those with light axial loads and light vertical reinforcement (Seible et al., 1994a, b).

b. Deformation

The deformation limit for sliding-shear behavior modes may be governed by the fracture of bars (dowels) crossing the sliding plane, crushing of the base course of masonry, or degradation of the shear and flexure transfer mechanisms in the flexural compression zone of the wall as the wall slides beyond its support.

6.3.5 Wall Instability

Out-of-plane buckling in the compression zone of flexural walls has been observed in experiments (Paulay and Priestley, 1992), but has not been reported for actual masonry structures subjected to earthquakes. The phenomenon is associated with compression stresses in flexural reinforcement that has achieved large inelastic tensile strains in previous cycles. Until the reinforcement yields in compression and the flexural cracks close, the reinforcement must carry the entire flexural compression force alone, thus leaving the wall in the compression zone vulnerable to buckling. Masonry walls, where the reinforcement is typically centered, are particularly vulnerable. Paulay and Priestley (1993) have suggested a simplified design relationship to calculate the critical wall width for which instability may limit ductility. For building evaluation, it is useful to determine the maximum ductility that may be expected for a given wall thickness. The following relationships, where the thickness, length, and height of the wall are given by t, l, and h, may be used to identify walls for which stability may be a limiting factor:

For $\frac{t}{l_w} \leq \frac{1}{24}$ or $\frac{t}{h_e} \leq \frac{1}{18}$
the displacement ductility may be no greater than $\mu_\Delta = 4$ (6-14)

For $\frac{t}{l_w} \geq \frac{1}{12}$ and $\frac{t}{h_e} \geq \frac{1}{8}$
the displacement ductility will not be limited by stability (6-15)

Experimental tests on slender masonry walls at large ductilities suggest that this relationship may be conservative (Seible et al., 1994a, b).

The lack of evidence for this type of failure in existing structures may be due to the large number of cycles at high ductility that must be achieved – most conventionally-designed masonry walls are likely to experience other behavior modes such as diagonal shear before instability becomes a problem.

6.3.6 Lap-Splice Slip

Relatively little research has been conducted specifically to investigate aspects of lap-splice slip that are unique to reinforced masonry as opposed to reinforced concrete (Hammons et al., 1994; Soric and Tulin, 1987). Experimental evidence of strength and/or deformation capacity of reinforced masonry components being limited by lap-splice slip failure has been noted in shear walls (Igarashi et al., 1993; Shing et al., 1991; Kubota and Murakami, 1988) and masonry beams (Okada and Kumazawa, 1987). Experimental studies in which lap splices were specifically avoided in plastic-hinge regions (Kingsley et al., 1994; Seible et al., 1994b; Shing et al., 1991) have shown superior performance over similar component tests including lap splices. In particular, the specimens without lap-splices in the plastic-hinge zones showed development of large curvature ductilities and well-distributed cracking in plastic hinge zones.

Research in Japan (for example, Seible et al., 1987; Okada and Kumazawa, 1987) has included the use of special spiral reinforcement for lap-splice confinement. Such reinforcement has been shown to limit successfully lap-splice slip, and to extend the effective plastic-hinge zone from one or two cracks to numerous cracks in the ends of masonry coupling beams or at the base of shear walls.

Studies to date indicate that lap splices in masonry are more susceptible to slip than are splices in concrete, because lap-splice regions in masonry are unlikely to include significant lateral confinement reinforcement. Hammons et al. (1994) found that splitting failure of masonry units in lap-splice regions was likely, regardless of lap-splice length, for bars #4 and greater in four-inch hollow units, #6 and greater in six-inch units, and #8 and greater in eight-inch units. While there are no experimental data on laps with more than two bars in a single grouted cell, it may be supposed that such bar configurations are susceptible to lap-splice failure.

For evaluation of lap-splice development length, l_d, refer to FEMA 222A, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings*, Section 8.4.5. The lap-splice equation in FEMA 222A should be modified to include the expected yield strength rather than the characteristic yield strength, and to use a strength-reduction factor of 1.0.

Rocking in reinforced masonry walls may develop as a consequence of lap-splice failure at the base of shear walls. While the strength of the wall can be compromised dramatically, there is evidence to suggest that rocking can be a stable mechanism of energy dissipation (Priestley et al.,1978; Igarashi et al.,1993). The validity of such a mechanism depends on the deformation capacity of connected components relative to the increased displacement demand that results from rocking.

6.3.7 Masonry Beams

Because of the physical restrictions of typical hollow clay or concrete masonry units, it is difficult to provide satisfactory confinement reinforcement, and impossible to provide diagonal reinforcement of masonry spandrels or coupling beams. It is therefore difficult to avoid preemptive shear or, at best, flexure/shear behavior modes in masonry beams. Masonry beams which are detailed to allow ductile flexural response are likely to fall under the category of RM3 components.

Bending and shear capacity of reinforced masonry beams may be evaluated using the principles set forth in FEMA 222A (BSSC, 1994), incorporating expected material strengths rather than characteristic strengths, and setting strength reduction factors equal to 1.0.

Many tests have been conducted on masonry beams under gravity loading, but few have been conducted under reversed cyclic loading with boundary conditions representative of typical coupled wall systems (i.e. incorporating slabs). A number of studies have been conducted in Japan as a part of the JTCCMAR research program, including Matsuno et al. (1987), Okada and Kumazawa (1987), and Yamazaki et al. (1988a and 1988b).

6.4 Symbols for Reinforced Masonry

A_g = Gross crossectional area of wall

A_{si} = Area of reinforcing bar i

A_v = Area of shear reinforcing bar

A_{vf} = Area of reinforcement crossing perpendicular to the sliding plane

a = Depth of the equivalent stress block

c = Depth to the neutral axis

C_m = Compression force in the masonry

f_{me} = Expected compressive strength of masonry

f_{ye} = Expected yield strength of reinforcement

h_e = Effective height of the wall (height to the resultant of the lateral force) = M/V

l_d = Lap splice development length

l_p = Effective plastic hinge length

l_w = Length of the wall

M/V = Ratio of moment to shear (shear span) at a section

M_e = Expected moment capacity of a masonry section

P_u = Wall axial load

s = Spacing of reinforcement

t = Wall thickness

V_e = Expected shear strength of a reinforced masonry wall

V_m = Portion of the expected shear strength of a wall attributed to masonry

V_s = Portion of the expected shear strength of a wall attributed to steel

V_p = Portion of the expected shear strength of a wall attributed to axial compression effects

V_{se} = Expected sliding shear strength of a masonry wall

x_i = Location of reinforcing bar i

Δ_p = Maximum inelastic displacement capacity

Δ_y = Displacement at first yield

ϕ_m = Maximum inelastic curvature of a masonry section

ϕ_y = Yield curvature of a masonry section

μ_Δ = Displacement ductility

μ = Coefficient of friction at the sliding plane

6.5 Reinforced Masonry Component Guides

The following Component Damage Classification Guides contain details of the behavior modes for reinforced masonry components. Included are the distinguishing characteristics of the specific behavior mode, the description of damage at various levels of severity, and performance restoration measures. Information may not be included in the Component Damage Classification Guides for certain damage severity levels; in these instances, for the behavior mode under consideration, it is not possible to make refined distinctions with regard to severity of damage. See also Section 3.5 for general discussion of the use of the Component Guides and Section 4.4.3 for information on the modeling and acceptability criteria for components.

RM1A	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Masonry**
		Component Type:	**Stronger Pier**
		Behavior Mode:	**Ductile Flexural**
		Applicable Materials:	**Fully grouted hollow concrete or clay units**

How to distinguish behavior mode:

By observation:

Damage in an RM1 component with a flexural response is likely to be localized in a zone with a vertical extent equal to approximately twice the length of the wall. Both horizontal and diagonal cracks of small size (< 0.05 in.) and uniform distribution may be present. Diagonal cracks typically propagate from horizontal, flexural cracks, and therefore have similar, regular spacing. If shear deformations are localized to one or two diagonal cracks of large width, the behavior mode is likely to be Flexure/Shear or Preemptive Shear. If a permanent horizontal offset is visible, the behavior mode may be Flexure/Sliding Shear

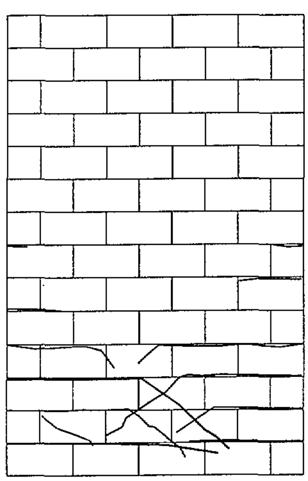
Caution: At low damage levels, damage observations will be similar to those for other behavior modes.

By analysis:

A wall detailed to ensure ductile flexural response will have sufficient horizontal reinforcement to allow development of a flexural plastic hinge mechanism through stable and distributed yielding of the vertical bars at the base of the wall. The ultimate capacity of the horizontal reinforcement alone in the hinge zone should be greater than the shear developed at the moment capacity of the wall. Wall vertical loads are likely to be small.

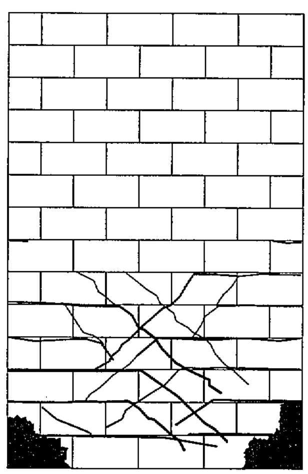
Refer to Evaluation Procedures for:

- Evaluation of flexural response.
- Identifying flexural versus shear cracks.
- Crack evaluation.

Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No crack widths exceed 1/16", and • No significant spalling *Typical Appearance:* 	Not necessary for restoration of structural performance. (Cosmetic measures may be necessary for restoration of nonstructural characteristics.)

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

RM1A

Severity	Description of Damage	Performance Restoration Measures
Slight $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No crack widths exceed 1/8" • No significant spalling or vertical cracking *Typical Appearance:* Similar to insignificant damage except cracks are wider and more extensive.	• Inject cracks $\lambda_K{}^* = 0.9$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Moderate $\lambda_K = 0.4$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$	*Criteria:* • Crack widths do not exceed 1/8" • Moderate spalling of masonry unit faceshells or vertical cracking at toe regions • No buckled or fractured reinforcement • No significant residual displacement. *Typical Appearance:* Similar to slight damage except cracks are wider and more extensive.	• Remove and patch spalled masonry and loose concrete. Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Extreme	*Criteria: Typical Indications* • Reinforcement has fractured. • Wide flexural cracking (>1/8" residual) • Large residual displacement • Extensive crushing or spalling • Visibly fractured or buckled reinforcing 	• Replacement or enhancement required.

RM1B	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Masonry**
		Component Type:	**Stronger Pier**
		Behavior Mode:	**Flexure / Shear**
		Applicable Materials:	**Fully grouted hollow concrete or clay units**

How to distinguish behavior mode:

By observation:

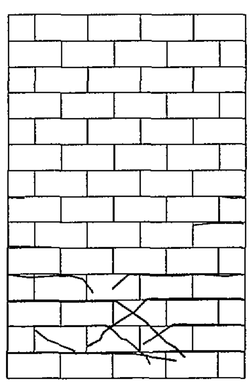
Damage in an RM1 component with a flexural /shear response is typically localized to the base of the wall, within the plastic hinge region. Both horizontal and diagonal cracks will be present, with diagonal cracks predominant. Diagonal cracks may appear to be independent from horizontal, flexural cracks, and may propagate across the major diagonal dimensions. At heavy damage levels, shear deformations are likely to be localized to one or two diagonal cracks of large width. If a permanent horizontal offset is visible, the behavior mode may be Flexure/Sliding Shear

By analysis:

Analysis of a wall with a Flexure / Shear behavior mode may be difficult, with no clear distinction between the controlling mechanism of flexure (deformation-controlled) or shear (force-controlled). Calculated capacities should be in the same range. Wall axial loads may be moderate-to-high.

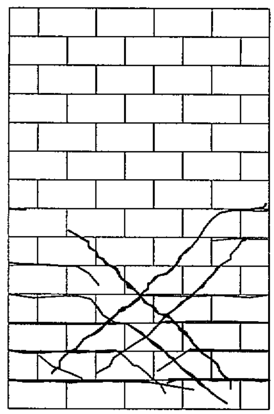
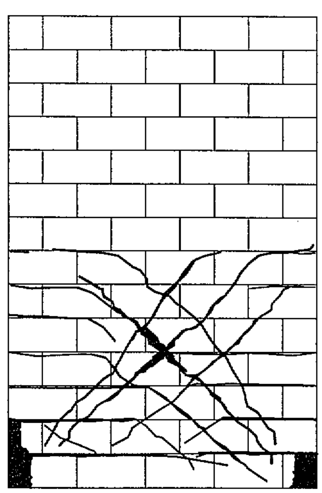
Refer to Evaluation Procedures for:

- Evaluation of flexural response.
- Evaluation of shear response
- Evaluation of plastic hinge length
- Identifying flexural versus shear cracks.
- Crack evaluation.

Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No crack widths exceed 1/16", and • No significant spalling *Typical Appearance:* 	Not necessary for restoration of structural performance. (Cosmetic measures may be necessary for restoration of nonstructural characteristics.)
Slight $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No crack widths exceed 1/8", and • No significant spalling or vertical cracking *Typical Appearance:* Similar to insignificant damage except cracks are wider with more extensive cracking.	• Inject cracks $\lambda_K{}^* = 0.9$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

RM1B

Severity	Description of Damage	Performance Restoration Measures
Moderate $\lambda_K = 0.4$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Criteria:* • Crack widths do not exceed 3/16" • Moderate spalling of masonry unit faceshells or vertical cracking at toe regions • No buckled or fractured reinforcement • No significant residual displacement *Typical Appearance:* 	• Remove and patch spalled masonry and loose concrete. • Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Extreme	*Criteria: Typical Indications* • Reinforcement has fractured • Wide flexural cracking (> ¼"), typically concentrated in a single crack • Wide diagonal cracking, typically concentrated in one or two cracks • Crushing or spalling at wall toes of more than one-half unit height or width, delamination of faceshells from grout • Visibly fractured or buckled reinforcing *Typical Appearance* 	• Replacement or enhancement required.

RM1C	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Masonry**
		Component Type:	**Stronger Pier**
		Behavior Mode:	**Flexure / Sliding Shear**
		Applicable Materials:	**Fully or partially grouted hollow concrete or clay units**

How to distinguish behavior mode:

By observation:

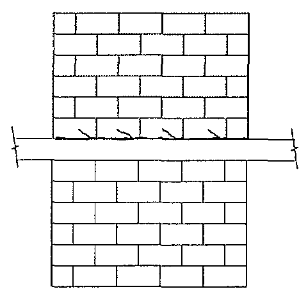
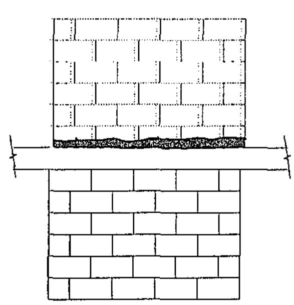
Evidence of movement will appear first in the form of pulverized mortar across a bed joint or construction joint. If grout cores include shear keys into the slab below, short diagonal cracks initiating at the keys may be visible in the course above the sliding joint. After severe sliding, crushing of the bottom course of masonry may occur.

By analysis:

Walls with very light axial loads ($P/f'_m A_g \leq 0.05$) may be susceptible to sliding, as are walls with very light flexural reinforcement, or large ductility demands.

Refer to Evaluation Procedures for:

- Evaluation of sliding response.

Severity	Description of Damage		Performance Restoration Measures
Insignificant		See **RM1A**	See **RM1A**
Slight $\lambda_K = 0.5$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$	*Typical Appearance*	As for **RM1A** or **RM1B** and: 	Not necessary for restoration of structural performance. (Cosmetic measures may be necessary for restoration of nonstructural characteristics.)
Moderate $\lambda_K = 0.2$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Typical Appearance*	Similar to slight with more extensive cracking and movement	• Remove and patch spalled masonry and loose concrete. • Inject cracks. $\lambda_K^* = 0.8$ $\lambda_Q^* = 1.0$ $\lambda_D^* = 1.0$
Extreme	*Criteria* *Typical Appearance* As for **RM1A** or **RM1B** and:	• Permanent wall offset • Spalling and crushing at base 	• Replacement or enhancement required.

RM1D	COMPONENT DAMAGE CLASSIFICATION GUIDE		
		System:	**Reinforced Masonry**
		Component Type:	**Stronger Pier**
		Behavior Mode:	**Flexure / Out-of-Plane Instability**
		Applicable Materials:	**Fully or partially grouted hollow concrete or clay units**

How to distinguish behavior mode:

By observation:

As for any unstable behavior mode, there will be little evidence of impending failure. Instability of the compression toe is preceded by large horizontal flexural cracks with significant plastic strains in the reinforcement crossing the crack. Evidence of such cracks, particularly when distributed across the plastic hinge zone rather than localized, may indicate incipient failure. Following failure, the wall will have visible out-of-plane displacements and localized crushing.

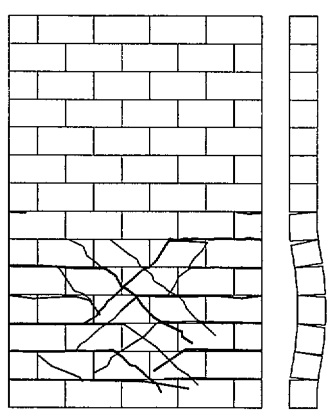
Caution: At low damage levels, damage observations will be identical to those for the **RM1A** behavior mode.

By analysis:

Walls with a tendency for compression toe instability will have large flexural displacement capacity, and little possibility for shear failure, even at large ductilities. Wall thickness will be less than or equal to the critical wall thickness for instability.

Refer to Evaluation Procedures for:

- Evaluation of flexural response.
- Evaluation of wall instability
- Identification of the plastic hinge zone.

Severity	Description of Damage	Performance Restoration Measures
Insignificant	See **RM1A**	See **RM1A**
Slight	See **RM1A**	See **RM1A**
Moderate	See **RM1A**	See **RM1A**
Heavy $\lambda_K = 0.4$ $\lambda_Q = 0.5$ $\lambda_D = 0.5$	*Typical Appearance* 	• Complete or partial replacement or enhancement required.
Extreme	*Criteria:* • Compression toe of wall buckled. • Reinforcement has fractured. • Wide flexural cracking. • Laterally displaced units. • Localized crushing or spalling.	• Replacement or enhancement required.

RM1E	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Masonry**
		Component Type:	**Stronger Pier**
		Behavior Mode:	**Flexure / lap splice slip**
		Applicable Materials:	**Fully or partially grouted hollow concrete or clay units**

How to distinguish behavior mode:

By observation:

Walls that are vulnerable to lap splice slip will exhibit flexural response, and possible flexure/shear response, until the lap splice capacity is exceeded. Observed damage will therefore be very similar to RM1A and RM1B until lap splice slip occurs. Lap splice slip failure is characterized by splitting of the masonry units parallel to the reinforcing bars.

Caution: At low damage levels, damage observations will be identical to those for the **RM1A** behavior modes.

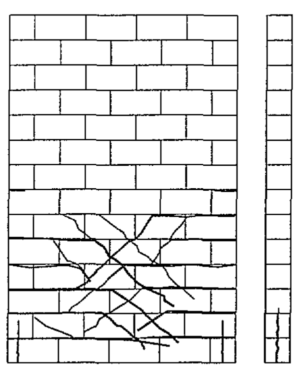
By analysis:

Walls that are vulnerable to lap splice slip may have:

- Bar size greater than:
  #4 in 4 inch units
  #6 in 6 inch units
  #8 in 8 inch units
- Lap splice less than l_d

Refer to Evaluation Procedures for:

- Evaluation of flexural response.
- Evaluation of lap splice slip response.
- Evaluation of development length l_d.

Severity	Description of Damage		Performance Restoration Measures
Insignificant		See **RM1A** or **RM1B**	See **RM1A** or **RM1B**
Slight		See **RM1A** or **RM1B**	See **RM1A** or **RM1B**
Moderate $\lambda_K = 0.4$ $\lambda_Q = 0.5$ $\lambda_D = 0.8$	*Criteria:* *Typical Appearance*	• Vertical cracks at toe of wall, particularly in narrow dimension of wall. 	See **RM1A** or **RM1B**

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

RM1E

Severity	Description of Damage		Performance Restoration Measures
Extreme	*Criteria:*	• Splitting of face shells at toe of wall • Crushing and delamination of faceshells from grout cores	• Replacement or enhancement required.
	Typical Appearance	• Wide flexural cracking and/or crushed units at base of wall • Pulverized mortar at base – evidence of rocking.	

RM2B	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Masonry**
		Component Type:	**Weaker Pier**
		Behavior Mode:	**Flexure / Shear**
		Applicable Materials:	**Fully grouted hollow concrete or clay units**

How to distinguish behavior mode:

By observation:

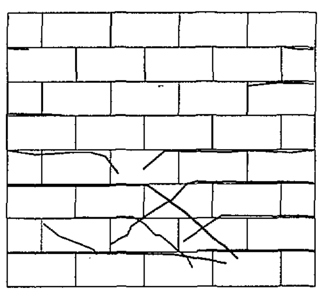
Damage in an RM2 component with a flexural/shear response may be localized to the first story, or it may be evident at a number of levels in story-height piers. Both horizontal and diagonal cracks may be present, with diagonal cracks predominant. Diagonal cracks may appear to be independent from horizontal flexural cracks, and propagate across the major diagonal dimensions. When severely damaged, shear deformations will be localized to one or two diagonal cracks of large width. If diagonal cracks are uniformly distributed and of small width, the behavior mode may be ductile flexure. If a permanent horizontal offset is visible, the behavior mode may include Flexure/ Sliding Shear.

By analysis:

Analysis of a wall with a Flexure / Shear behavior mode may not indicate a clear distinction between the controlling mechanism of flexure (deformation controlled) or shear (force controlled). Calculated capacities should be in the same range. Wall axial loads may be moderate to high.

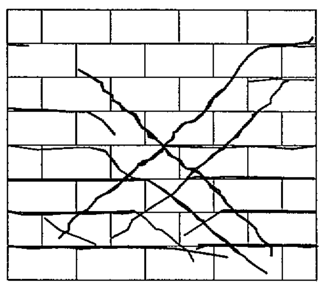
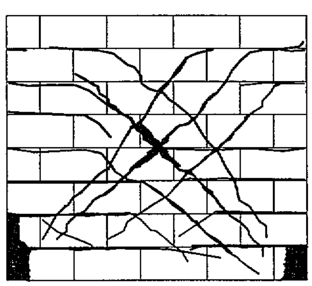
Refer to Evaluation Procedures for:

- Evaluation of flexural response.
- Evaluation of shear response.
- Identifying flexural versus shear cracks.
- Crack width discussion.

Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No crack widths exceed 1/16." • No significant spalling. *Typical Appearance:*  May appear similar to flexure following small displacement cycles. Diagonal cracks often propagate from horizontal cracks.	Not necessary for restoration of structural performance. (Cosmetic measures may be necessary for restoration of nonstructural characteristics.)

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

RM2B

Severity	**Description of Damage**	**Performance Restoration Measures**
Slight $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No crack widths exceed 1/8". • No significant spalling or vertical cracking. *Typical Appearance:* Similar to insignificant damage, except cracks are wider and cracking is more extensive.	• Inject cracks. $\lambda_K{}^* = 0.9$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Moderate $\lambda_K = 0.4$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Criteria:* • Crack widths do not exceed 3/16". • Moderate spalling of masonry unit faceshells or vertical cracking at toe regions. • No buckled or fractured reinforcement. • No significant residual displacement. *Typical Appearance:* 	• Remove and patch spalled masonry and loose concrete. Inject cracks. • Consider horizontal fiber composite overlay. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Extreme	*Criteria:* • Reinforcement has fractured *Typical Indications* • Wide flexural cracking typically > ¼" concentrated in a single crack. • Wide diagonal cracking, typically concentrated in one or two cracks • Extensive crushing or spalling at wall toes, visible delamination of faceshells from grout *Typical Appearance* 	• Replacement or extensive enhancement required.

RM2G	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Masonry**
		Component Type:	**Weaker Pier**
		Behavior Mode:	**Preemptive Shear**
		Applicable Materials:	**Fully grouted hollow concrete or clay units**

How to distinguish behavior mode:

By observation:

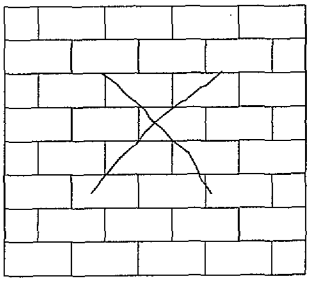
At low levels of damage, wall may appear similar to RM2B. Diagonal cracks may be visible before flexural cracks. Damage occurs quickly in the form of one or two dominant diagonal cracks. Subsequent cycles may cause crushing or face shell debonding at the center of the wall and/or at the wall toes.

By analysis:

Calculated shear load capacity, including both masonry and steel components, will be less than or equal to shear associated with flexural load capacity

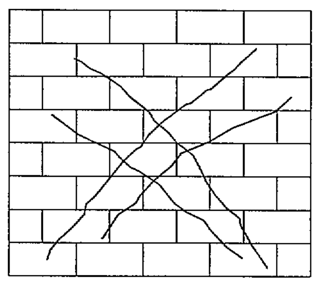
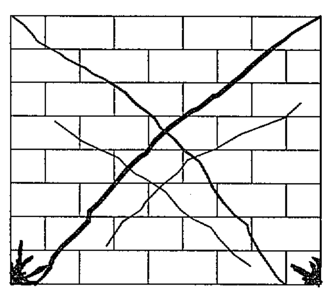
Refer to Evaluation Procedures for:

- Evaluation of flexural response.
- Evaluation of shear response.
- Evaluation of crack patterns.
- Crack evaluation.

Severity	**Description of Damage**	**Performance Restoration Measures**
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No diagonal cracks. • Flexural crack <1/16". • No significant spalling. *Typical Appearance:* No visible damage.	Not necessary for restoration of structural performance. (Cosmetic measures may be necessary for restoration of nonstructural characteristics.)
Slight $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No crack widths exceed 1/16". • No significant spalling or vertical cracking. *Typical Appearance:* Similar to insignificant damage, except that small diagonal cracks may be present. 	• Inject cracks. $\lambda_K^* = 0.9$ $\lambda_Q^* = 1.0$ $\lambda_D^* = 1.0$

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

RM2G

Severity	Description of Damage	Performance Restoration Measures
Moderate $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Criteria:* • Crack widths do not exceed 1/16". • No spalling of masonry unit faceshells or vertical cracking at toe regions. *Typical Appearance:* May be several diagonal cracks, typically with one dominant crack. 	• Inject cracks. • Consider horizontally oriented fiber composite overlay. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Heavy $\lambda_K = 0.3$ $\lambda_Q = 0.4$ $\lambda_D = 0.5$ See FEMA 307 for calculation of λ_Q	*Criteria:* • Single dominant crack, may be > 3/8". *Typical Appearance:* 	• Inject cracks. • Provide horizontally oriented fiber composite overlay. • Consider replacement.
Extreme	*Criteria:* • Reinforcement has fractured. *Typical Indications* • Wide diagonal cracking, typically concentrated in one or two cracks. • Crushing or spalling at center of wall or at wall toes.	• Replacement or enhancement required.

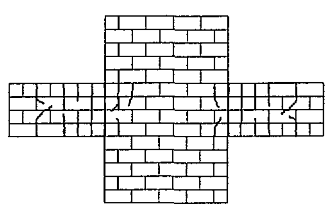
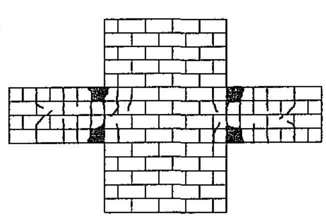
RM3A	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Masonry**
		Component Type:	**Weaker Spandrel**
		Behavior Mode:	**Flexure**
		Applicable Materials:	**Fully grouted hollow concrete or clay units**

How to distinguish behavior mode:

By observation:	*By analysis:*
Masonry wall frames will develop numerous flexural cracks within the beam plastic hinge zones, with very little damage in the pier or joint regions. If significant damage develops in piers, component should be reclassified as RM2.	Wall frame dimensions and reinforcement satisfy the requirements of Section 2108.2.6 of the 1994 or 1997 *UBC*, <u>or</u> the component can be shown by the principles of capacity design to develop flexural plastic hinges in the beams without developing the strength of the piers.

Refer to Evaluation Procedures for:

- Evaluation of flexural response.
- Identification of the plastic hinge zone.

Severity	Description of Damage		Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:*	• No crack widths exceed 1/16". • No significant spalling.	Not necessary for restoration of structural performance. (Cosmetic measures may be necessary for restoration of nonstructural characteristics.)
Slight $\lambda_K = 0.8$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$	*Criteria:* *Typical Appearance:*	• No crack widths exceed 1/8". • No significant spalling. 	• Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Moderate $\lambda_K = 0.6$ $\lambda_Q = 0.8$ $\lambda_D = 1.0$	*Criteria:* *Typical Appearance:*	• No crack widths exceed ¼". • Minor spalling (less than one unit depth) in beam ends. 	• Replace spalled material. • Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Extreme	*Criteria:* *Typical Appearance*	• Reinforcement has fractured. • Wide flexural cracking (> 3/8"). • Significant crushing or spalling at junction of pier and beams.	• Replacement or enhancement required.

RM3G	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Masonry**
		Component Type:	**Weaker Spandrel**
		Behavior Mode:	**Preemptive Shear**
		Applicable Materials:	**Fully or partially grouted hollow concrete or clay units**

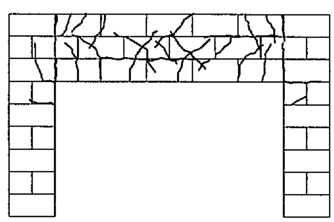
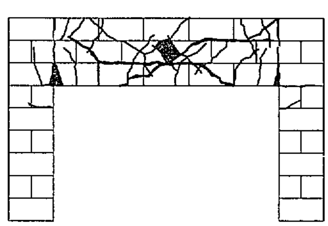
How to distinguish behavior mode:

By observation:
Cracking in reinforced masonry beams may be concentrated at the beam ends or distributed over the beam. Development of a plastic hinge zone is unlikely because of the difficulty of providing sufficient confinement reinforcement. Visible cracking is often a continuation of cracks in slabs or other adjacent elements, so beam damage should be evaluated in the context of the system behavior. If damage patterns appear to be associated with ductile flexural response, component may be reclassified as RM3.

By analysis:
Expected shear strength will typically be less than shear associated with the development of a flexural yielding mechanism at each end of the beam. If a stable plastic hinge can develop, component may be reclassified as RM3.

<u>*Refer to Evaluation Procedures for:*</u>

- Evaluation of flexural response.

Severity	Description of Damage		Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria*	• Hairline cracks only.	Not necessary for restoration of structural performance. (Cosmetic measures may be necessary for restoration of nonstructural characteristics.)
Moderate $\lambda_K = 0.8$ $\lambda_Q = 0.8$ $\lambda_D = 1.0$	*Criteria* *Typical Appearance*	• Cracks < 1/8". 	• Inject cracks. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Heavy $\lambda_K = 0.3$ $\lambda_Q = 0.5$ $\lambda_D = 0.9$	*Criteria* *Typical Appearance*	• Cracks > 1/8". 	• Inject cracks. • Repair spalled areas $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Extreme	*Criteria:* *Typical Appearance*	• Reinforcement has fractured. • Crushing or spalling at beam ends. • Large diagonal cracks and / or spalling at center portion of beam.	• Replacement or enhancement required.

7: Unreinforced Masonry

7.1 Introduction and Background

7.1.1 Section Organization

This section summarizes evaluation methodologies and repair recommendations for earthquake-damaged unreinforced masonry (URM) bearing-wall buildings. Reinforced masonry is covered in Chapter 6. Masonry with less than 25 percent of the minimum reinforcement required by FEMA 273 (ATC, 1997a) should be considered unreinforced. This material supports and supplements the Component Damage Classification Guides (Component Guides) for URM components contained in Section 7.5. The section is organized as follows:

Section 7.1 discusses the various materials and structural systems used in URM buildings, evaluation of rehabilitated buildings, and the limitations of the URM guidelines.

Section 7.2 identifies typical URM elements, components, and behavior modes, as well as characteristics of the types of earthquake damage these components can experience. Behavior modes discussed include those resulting from in-plane and out-of-plane demands on walls and those occurring in other elements or due to the interrelationships between building elements. Information on the relative likelihood of occurrence of each damage type is included, where information is available. For in-plane behavior modes, the strength and displacement capacity of each mode is discussed along with uncertainties in capacity calculations.

Section 7.3 presents evaluation procedures for URM walls subjected to in-plane and out-of-plane demands. This section also identifies the testing that may be needed to provide information on required material properties. Symbols are listed in Section 7.4 and references are listed in the Reference section.

FEMA 307 provides a summary of the hysteretic behavior observed in experimental tests of URM specimens and commentary on the FEMA 273 force-displacement relationships. It describes the development of the λ-factors in the component guides of FEMA 306. FEMA 307 also provides a tabular summary of important experimental research and a list of other references on URM elements.

7.1.2 Material Types and Structural Framing

Unreinforced masonry is one of the oldest and most diverse building materials. Important material variables include masonry unit type, construction, and the material properties of various constituents.

Solid clay-brick unit masonry is the most common type of masonry unit, but there are a number of other common types, such as hollow clay brick, structural clay tile, concrete masonry, stone masonry, and adobe. There are additional subgroupings within each of these larger categories. For example, as shown in FEMA 274, structural clay tile has been classified into structural clay load-bearing wall tile, structural clay non-load-bearing tile (used for partitions, furring, and fireproofing), structural clay floor tile, structural clay facing tile, and structural glazed facing tile. Hollow clay tile (HCT) is a more common term for some types of structural clay tile. Concrete masonry units (CMU) can be ungrouted, partially grouted, or fully grouted. Stone masonry can be made from any type of stone, but sandstone, limestone, and granite are common. Other stones common in a local area are used as well. Sometimes materials are combined, such as brick facing over CMU backing, or stone facing over a brick backing.

Wall construction patterns also vary widely, with bond patterns ranging from common running bond in brick to random ashlar patterns in stone masonry to stacked bond in CMU buildings. The variety of solid brick bond patterns is extensive. Key differences include the extent of header courses, whether collar joints are filled, whether cavity-wall construction was used, and the nature of ties between facing and backing wythes. In the United States, for example, typical running-bond brick masonry includes header courses interspersed by about five to six stretcher courses. Header courses help tie the wall together and allow it to behave in a more monolithic fashion for both in-plane and out-of-plane demands. The UCBC (ICBO, 1994) has specific prescriptive requirements on the percentage, spacing, and depth of headers. Facing wythes not meeting these requirements must be considered as veneer and are therefore not used to determine the effective thickness of the wall. Veneer wythes must be tied to the backing to help prevent out-of-plane separation and falling hazards. Although bed and head joints are routinely filled with mortar, the extent of collar-joint fill varies widely. Completely filled collar joints with metal ties